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THE REGIONAL MUNICIPALITY OF HAMILTON-WENTWORTH

POLLUTION CONTROL PLAN

A JOINT STUDY BY:

paul theil associates limited
beak consultants limited

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REGIONAL MUNICIPALITY OF HAMILTON-WENTWORTH

POLLUTION CONTROL PLAN

December 1991

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in the preparation of this report.

EXECUTIVE SUMMARY

INTRODUCTION AND BACKGROUND

The Regional Municipality of Hamilton–Wentworth and the Ontario Ministry of the Environment initiated the Regional Municipality of Hamilton–Wentworth Pollution Control Plan (RMHWPCP) to develop a strategy for reducing pollutant loads to the local receiving water bodies. Conclusions and recommendations from the RMHWPCP may be used to assist in the development of a Remedial Action Plan (RAP) which, when implemented, will improve the water quality in the local receiving waters. The RAP will be a joint effort between the Region, Federal and Provincial agencies, and stakeholders.

The study area for the RMHWPCP is the Regional Municipality of Hamilton–Wentworth, with the core study area being the City of Hamilton. Within the study area, four major receiving water bodies exist, these are:

- Hamilton Harbour;
- Chedoke Creek;
- Cootes Paradise; and
- Redhill Creek.

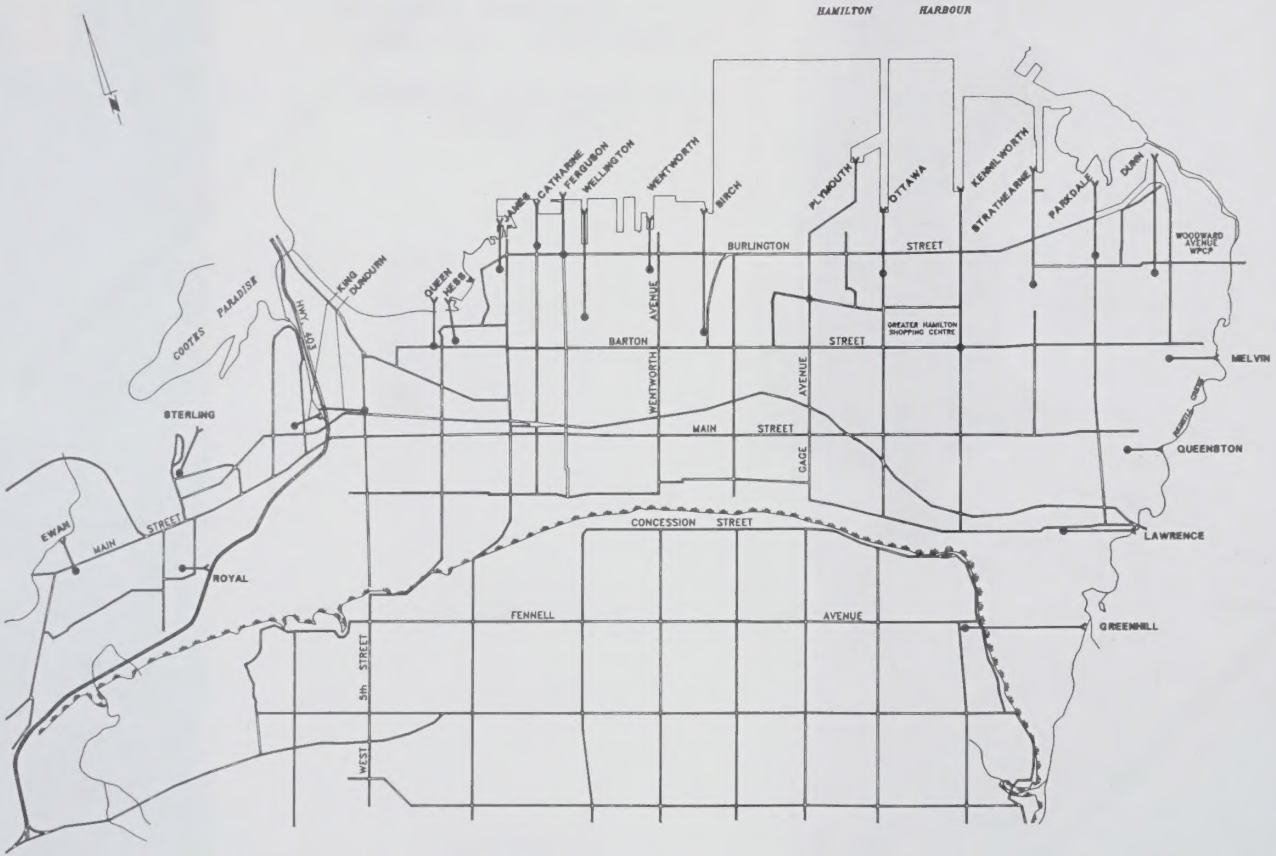
Chedoke Creek, Cootes Paradise and Redhill Creek discharge to Hamilton Harbour. Hamilton Harbour discharges to Lake Ontario.

Sources of contaminated discharges to the receiving water bodies are

- combined sewer overflows (CSO's) through 22 major CSO outfalls (Figure 1);
- water pollution control plant (WPCP) bypasses and effluent; and
- separated stormwater system discharges.

Each of these sources were considered in the study, although CSO's and WPCP discharges were the main items of concern. Both existing and future development conditions were considered in the study (Figure 2).

The quantity of CSO's discharging to the receiving bodies of water is largely determined by CSO flow regulators and the capacity of the sanitary interceptor sewers to convey combined sewage to the WPCP for treatment.



REGIONAL MUNICIPALITY OF HAMILTON - WENTWORTH

POLLUTION CONTROL PLAN

LEGEND

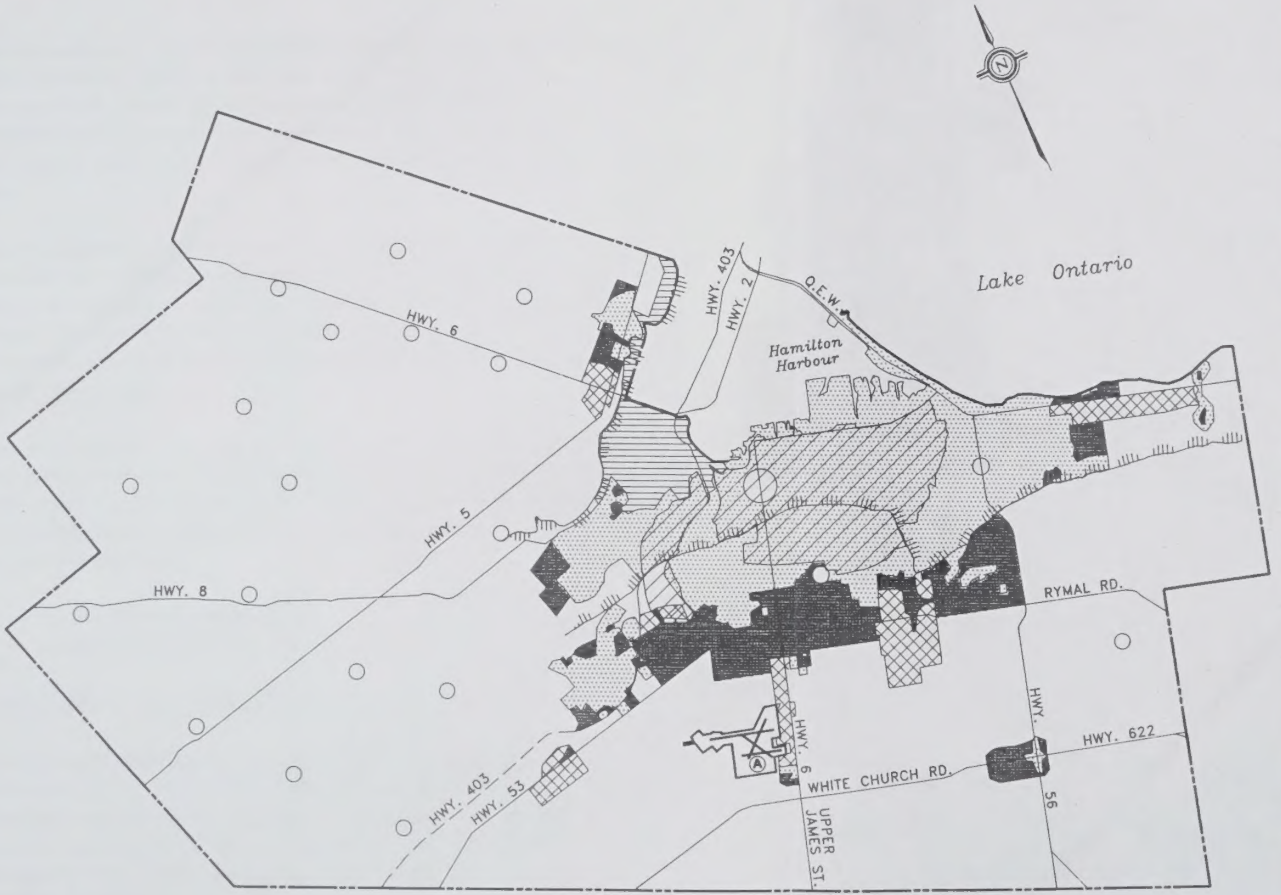
CSO Outfall

MAJOR CSO OUTFALLS AND RECEIVING WATERS IN THE HAMILTON AREA

Figure 2

REGIONAL MUNICIPALITY OF HAMILTON - WENTWORTH

EXISTING AND PROPOSED LAND USES



LEGEND

- Existing Development
- Proposed Industrial–Business Parks
- Proposed Residential and Related Uses
- Combined Sewer Area
- Parkway Belt West Policy Areas
- Rural Area
- Rural Settlements
- Niagara Escarpment
- Airport
- Regional Boundaries

1200 0 1200 2400 4800m

paul theil associates limited

Source: The Regional Development Pattern (1989)

CSO flow regulators are hydraulic structures located throughout the Region's combined sewer system which determine the quantity of water discharged to either the CSO outfalls or to the sanitary interceptor/WPCP. Approximately 50 percent of the Hamilton area is serviced by combined sewers, and within this area in excess of 100 flow regulators exist. This study identified approximately 35 CSO flow regulators which have significant effect on the response of the sewer system during rainfall events.

Two sanitary interceptor sewers service the Region. Both interceptors convey combined sewage to the Region's main WPCP located on Woodward Avenue. The western interceptor conveys combined sewage from West Hamilton and the central business district and, therefore, services the majority of the Region's population. The Redhill Creek interceptor conveys sewage from the mountain areas and Stoney Creek to the WPCP. Most sanitary flows from future development will be directed through the Redhill Creek interceptor.

Most of the population base within the Region is serviced by the Woodward Avenue WPCP. Approximately 380,000 are currently served by the WPCP and it is expected to service 500,000 by 2025. This facility, which consists of a conventional activated sludge treatment systems with phosphorous removal, treats approximately 303 million litres per day (MLD). Peak treatment rates at the plant are approximately:

- 409 MLD for primary treatment; and
- 545 MLD (long term) to 615 MLD (short term) for secondary treatment.

Smaller WPCP's also exist in Dundas and Waterdown to service their respective communities.

Storm sewer services currently drain approximately 50 percent of the developed lands which discharge to Hamilton Harbour. Under future conditions, approximately two–thirds of the developed lands discharging to Hamilton Harbour will be serviced by separated stormwater services. The increase in the percent of lands serviced by separated sewers is the result of increased development, rather than replacement of the combined sewer services. Storm sewer services will be most common in the Redhill and Ancaster/Sulpher Creek areas.

STUDY OBJECTIVES

The objectives of this study are outlined below.

1. Ascertain the constituent loads to receiving water bodies from CSO's, WPCP discharges, and separated stormwater discharges. The constituents of interest are:

   - fecal coliform (FC)
   - total solids (TS)
   - suspended solids (SS)
   - 5–day biochemical oxygen demand (BOD_5)
   - total phosphorus (TP)
   - ammonia (NH_3)
   - lead (Pb)
   - copper (Cu)
   - Zinc (Zn)
   - total kjeldahl nitrogen (TKN).

2. Based on the loading results, prioritize the pollutant sources.

3. Develop and evaluate remedial strategies such that a range of pollutant load reductions may be attained. Emphasis will be placed on CSO's and maintaining the current effluent quality at the Woodward Avenue WPCP. The remedial measures will consider both existing and future development conditions.

4. Evaluate the cost effectiveness, feasibility and implementation considerations of proposed remedial measures.

These objectives were met by:

- reviewing existing reports, plans, and data;
- initiating data acquisition programs;

3. Under dry weather flow conditions, approximately 20 percent of the capacity, and storage, of the western interceptor is used. Under wet weather flow conditions, the majority of the western interceptor is normally full and surcharged for rainfall events greater than 15 mm in depth.

4. Without the capability to close the WPCP controlled sluice gates within the City of Hamilton during rainfall events, thereby restricting inflows to sanitary interceptor sewers, the WPCP wet–well pumphouse may flood since the pumphouse capacity is less than interceptor sewer capacity.

5. During rainfall events, the Woodward Avenue WPCP is currently operated to maximize the volume of combined sewage which is treated. However, the primary treatment system is limited in that it can normally treat only 10 to 20 percent more sewage than dry weather flow conditions. This limitation requires bypassing excess flow around the primary treatment system. Primary bypassing is normally required for rainfall events greater than 5 mm in depth.

   The secondary treatment system has the capacity to treat approximately 50 (long–term) to 100 (short–term) percent more sewage than dry weather flow conditions. Secondary bypassing is normally required for rainfall events greater than 10 mm.

SYSTEM ANALYSIS

Combined Sewer Overflow Analysis

A combined sewer overflow analysis was conducted using a calibrated/validated SWMM4 (RUNOFF and TRANSPORT blocks) model of the system to ascertain CSO frequencies, volumes and trends. The analysis concentrated on existing sewershed conditions since the sewershed is essentially 100 percent developed and is not expected to change significantly in the future. The key steps in the analysis are outlined below:

1. The system's response to typical and intense rainfall events was ascertained. These rainfall events range from 5 mm to 35 mm in depth. A 5 mm event is representative of a typical storm within the Region and may be expected to occur once every one to two weeks. A 35 mm storm was the largest rainfall event considered in the analysis. A storm of this magnitude is expected to occur, on average, once each year.

2. System response was also determined during a dry, typical and wet year (defined as May 1 to October 31). The years selected for this analysis included:

   - 1971, a dry year with 294 mm of rainfall;
   - 1989, a typical year with 454 mm of rainfall;
   - 1981, a wet year with 706 mm of rainfall.

   On average, the Hamilton area receives approximately 490 mm of rainfall during the May 1 to October 31 period. Winter months were excluded from the analysis as the analysis of snowmelt events was beyond the scope of this study.

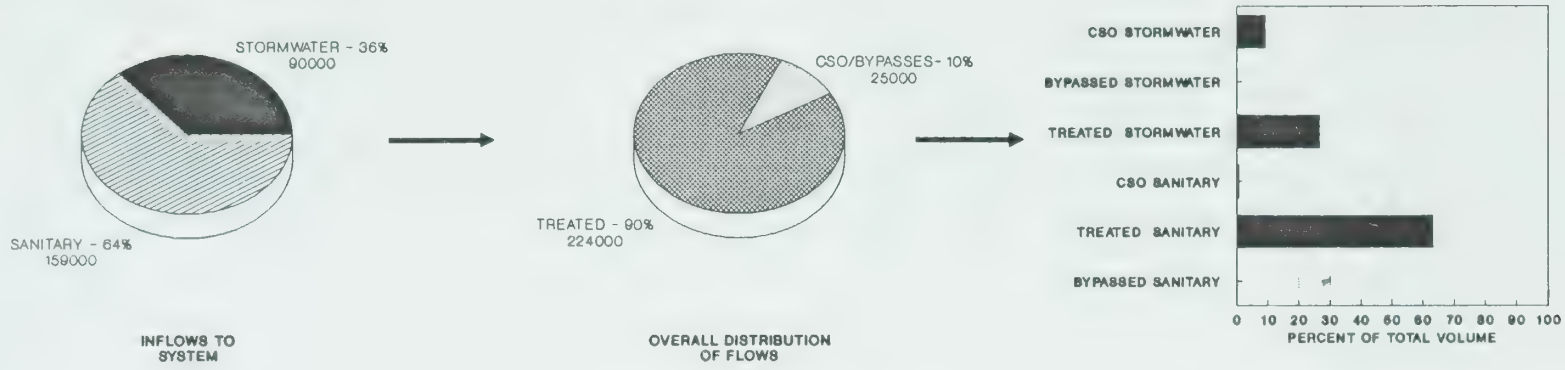
Conclusions which may be drawn from the CSO analysis are outlined below.

1. The existing combined sewer system and WPCP has sufficient capacity to convey and treat approximately 90 percent of all inflows to the systems for a 5 mm event. Conversely, approximately 75 percent of all inflows during a 2–hour 35 mm event (i.e. the 1 CSO/year design storm) will be routed to the receiving water bodies with little or no treatment. These results are illustrated in Figure 3. Substantial remedial works are, therefore, required to attain a high level of CSO control.

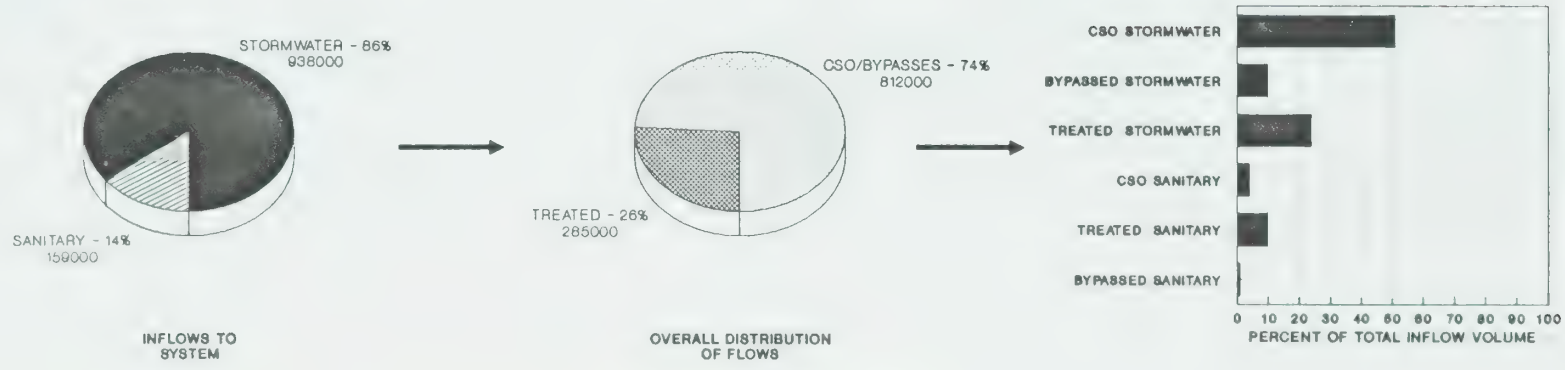
2. CSO volumes to the receiving water bodies are 2.4, 4.4 and 9.7 million cubic metres during the dry, typical and wet years, respectively. These volumes represent approximately 5 to 15 percent of the volume which enters the Region's combined sewer system annually (Figure 4).

3. On average, CSO's occur at the 22 major outfalls once every 8 days or 23 times during the May 1 to October 31 period (Figure 5). Approximately 15, 38, 56 and 85 percent of the annual CSO volume is discharged from the top 1, 3, 5 and 10 outfalls, respectively. Therefore, half of the CSO outfalls contribute less than 15 percent of the annual CSO volume.

FIGURE 3: DISTRIBUTION OF SANITARY AND STORMWATER FLOWS FOR 5 AND 35 mm EVENTS



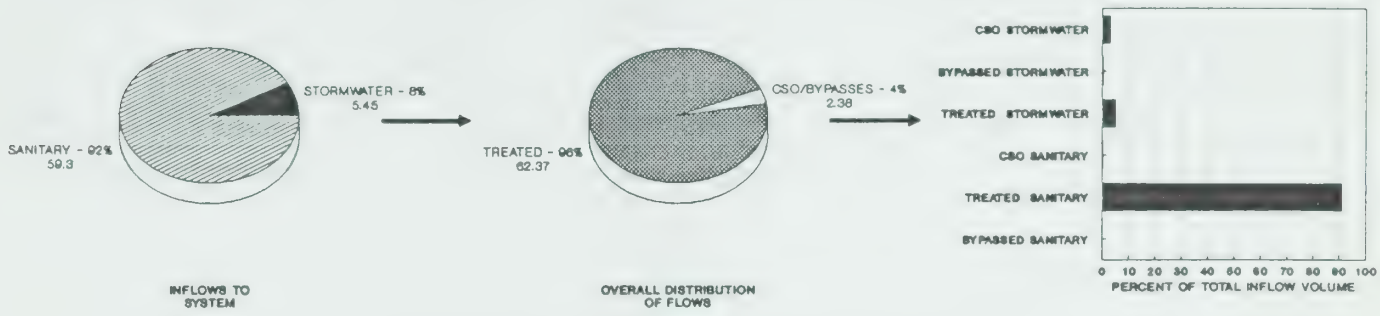
a) 5mm RAINFALL EVENT



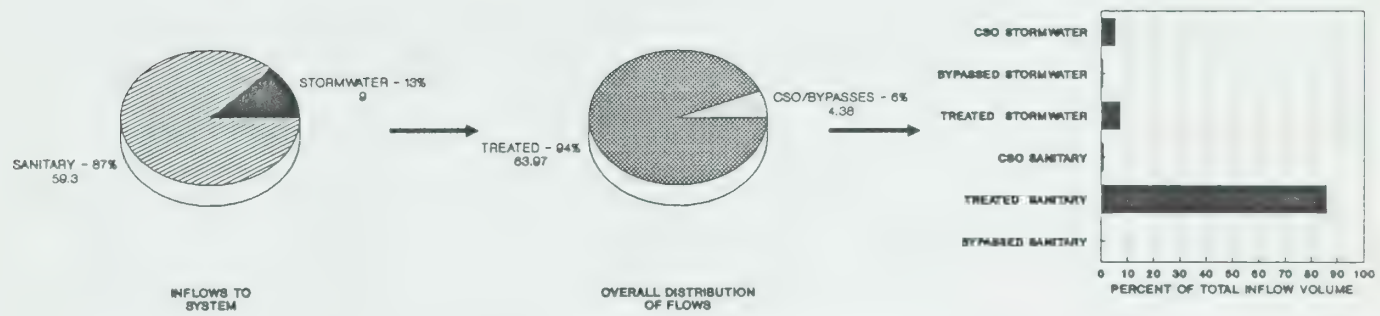
b) 35 mm RAINFALL EVENT

NOTE: VALUES UNDER FLOW LABELS ARE VOLUMES (CUBIC METRES)

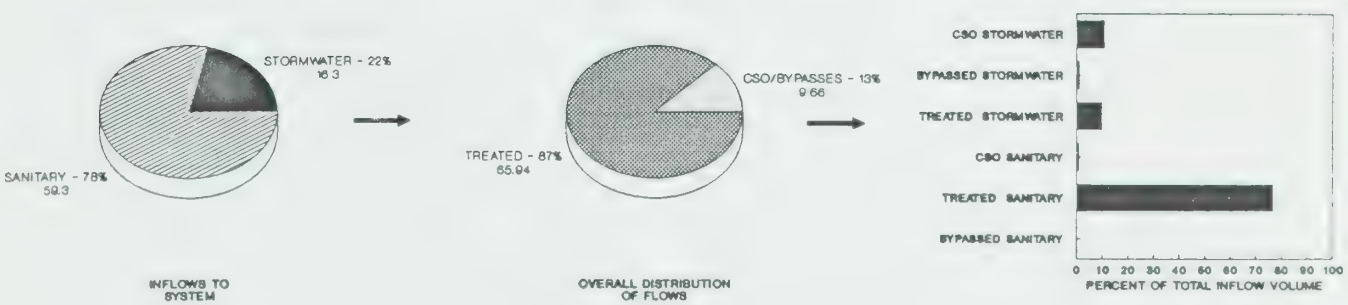
FIGURE 4: DISTRIBUTION OF SANITARY AND STORMWATER FLOWS FOR DRY, TYPICAL AND WET YEARS



a) DRY YEAR - 1971 - 294 mm RAINFALL



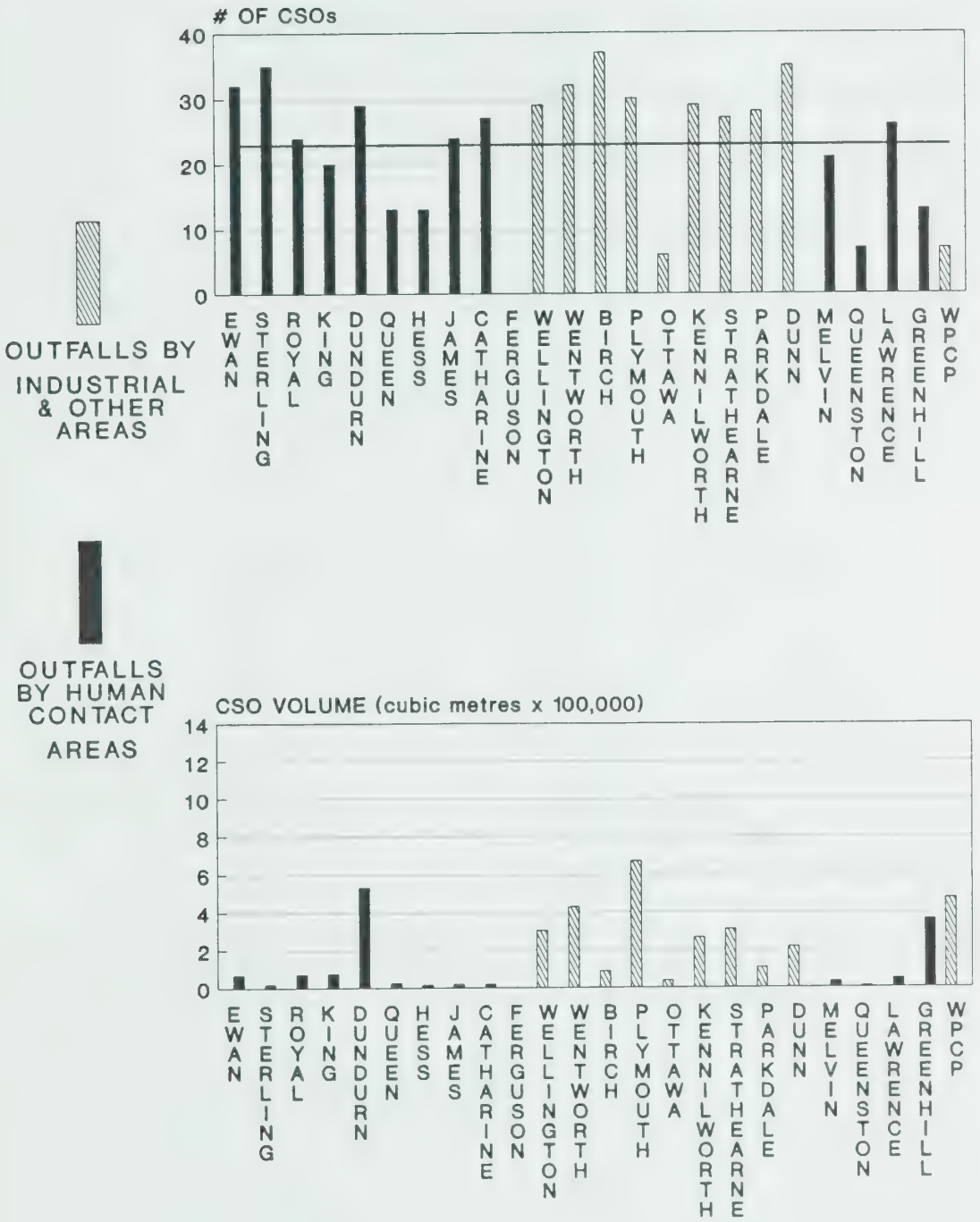
b) TYPICAL YEAR - 1989 - 454 mm RAINFALL



c) WET YEAR - 1981 - 706 mm RAINFALL

NOTE: VALUES UNDER FLOW LABELS ARE VOLUMES IN MILLIONS OF CUBIC METRES

FIGURE 5: CSO FREQUENCIES AND VOLUMES DURING A TYPICAL (1989) YEAR



4. Approximately one–third of the annual CSO volume is discharged through outfalls located by human contact areas. The remaining CSO volume is generally discharged in the east end industrial areas (Figure 5).

5. CSO's, on the average, are composed of 7 percent sanitary sewage and 93 percent stormwater sewage.

Woodward Avenue WPCP Analysis

The WPCP analysis concentrated on determining the required level of expansion at the Woodward Avenue WPCP in order to:

- accommodate a population base of 500,000 by the year 2025;
- treat the estimated CSO volumes for a given condition; and
- ensure that the current level of effluent quality is maintained.

Various WPCP process models were used in the analysis in order to ascertain the expansion requirements. These models accounted for the dynamic nature of combined sewer system inflows and allowed numerous expansion alternatives to be tested. A costing model, CAPDET, was also used in the analysis to ascertain the costs associated with WPCP expansion. Expansion options are discussed later.

Stormwater Runoff Analysis

A stormwater runoff analysis of the separated sewer areas draining to Hamilton Harbour/Cootes Paradise was conducted using a mass balance spreadsheet model. The emphasis of the analysis was to ascertain constituent loadings to Hamilton Harbour/Cootes Paradise:

- under existing conditions; and
- under future development conditions (Figure 2).

Key results are summarized in Table 1. Generally, future developments in the Hamilton area are expected to increase constituent loadings from separated stormwater systems to Hamilton

TABLE 1

SEPARATED STORM SEWER LOADINGS

TO HAMILTON HARBOUR/COOTES PARADISE

	Daily Loading (kg/day) for Specified Conditions (May 1 to October 31)		
Parameter	Existing	Future Conditions No Remedial Works	Percent Increase In Daily Loading
SS	6,000	8,200	37
TS	27,000	37,000	38
TP	20	27	35
NH_3	21	25	19
TKN	86	111	29
Zn	9	13	44
FC (total counts)	$1.4x10^{13}$	$2.10x10^{13}$	50
BOD_5	500	740	48
Pb	7.7	11	43
Cu	1.9	2.8	47

Harbour/Cootes Paradise by 40 percent. The increase in runoff volume (i.e. due to increases in impervious areas) is the primary reason why constituent loadings increase.

TOTAL LOADINGS TO HAMILTON HARBOUR/COOTES PARADISE

Total constituent loadings to Hamilton Harbour/Cootes Paradise were estimated for flows originating from:

- CSO's
- WPCP discharges; and
- separated storm sewer discharges.

Both existing and future development conditions were included in the analysis. Future development conditions consider:

- a population of 500,000 is serviced by the Woodward Avenue WPCP; and
- future development will be concentrated in the areas specified in Figure 2.

Results are included in Table 2 and outlined below.

Under existing conditions:

- the WPCP is the major source of loadings of SS, TS, NH_3, TP, BOD_5, Cu and TKN;
- CSO's are the main source of FC; and
- storm sewer discharges contribute the majority of Pb (lead) and Zn (Zinc) to the receiving waters.

On average, CSO's, stormwater discharges and WPCP discharges account for 18, 24 and 58 percent of the total loadings to Hamilton Harbour/Cootes Paradise, respectively.

With the projected population and development patterns:

- CSO loadings are assumed to remain the same as existing conditions since the combined sewer areas are essentially fully developed under existing conditions;

TABLE 2

TOTAL CONSTITUENT LOADINGS

TO HAMILTON HARBOUR/COOTES PARADISE

(MAY 1 TO OCTOBER 31)

Parameter	Daily Loading (kg/d) for Existing				Daily Loading (kg/d) for Future			
	CSO	WPCP	Storm	Total	CSO	WPCP	Storm	Total
SS	2,700	7,000	6,000	16,000	2,700	9,035	8,200	20,000
TS	8,500	150,000	27,000	186,000	8,500	195,000	37,000	240,000
TP	17	140	20	177	17	182	27	226
NH_3	30	2,086	21	2,140	30	2,710	25	2,765
TKN	76	3,058	86	3,220	76	3,975	111	4,160
Zn	7.9	3.1	9	20	7.9	4.0	13	25
FC (total counts)	$2.2x10^{13}$	$5.5x10^{11}$	$1.4x10^{13}$	$3.6x10^{13}$	$2.2x10^{13}$	$7.0x10^{11}$	$2.1x10^{13}$	$4.4x10^{13}$
BOD_5	410	2,800	500	3,700	410	3,610	740	4,755
Pb	3.3	2.1	7.7	13	3.3	2.7	11	17
Cu	0.9	6.3	1.9	9	0.9	8.2	2.8	12

Note: These estimated loadings are based on a planning level analysis of the Region's combined sewer system, stormwater drainage network, and WPCP discharges. CSO loadings are the most difficult loadings to estimate as they are influenced by numerous conditions/processes. See Appendix I for additional information regarding CSO loadings.

- WPCP loadings are expected to increase by approximately 30 percent due to increases in sanitary sewage inflows; and
- loadings from separated stormwater areas increase by approximately 40 percent as indicated in Table 1.

Overall, total loadings to Hamilton Harbour/Cootes Paradise are expected to increase approximately 28 percent under future conditions. Average contributions from CSO's, stormwater discharges and WPCP discharges to the total loadings are 14, 27 and 59 percent, respectively.

In summary, the Woodward Avenue WPCP is, and expected to remain, the major contributor of constituents to Hamilton Harbour/Cootes Paradise, assuming the effluent quality at the plant remains unchanged. The large contribution from the WPCP to the total constituent loadings is a result of:

- the continuous operation of the WPCP compared to the intermittent nature of CSO's (i.e. once every eight days or so);
- the higher concentrations of many of the water quality constituents in sanitary sewage; and
- the fact that the sanitary sewershed draining to the Woodward Avenue WPCP is approximately 20 percent larger than the combined sewershed under existing conditions, increasing to about 100 percent under future development conditions, as it contains separated sewer areas such as Ancaster and Stoney Creek.

Therefore, although remedial works to reduce CSO's will substantially decrease CSO loadings, they will only decrease total loadings to the Harbour marginally, since CSO's are not the primary sources of most constituents. That is, substantial reduction in constituent loadings will require the improvement of the WPCP effluent quality.

SELECTION OF CRITERIA FOR REDUCTION OF CONSTITUENT LOADINGS TO THE RECEIVING BODIES OF WATER

Target levels of reduction were set to provide objectives for the reduction of constituent loads to the receiving water bodies. Target levels for the reduction of constituent loads were required as:

- they impact the type of remedial alternatives which should be evaluated;

- they provide a framework for defining the feasibility of achieving the target; and
- they provide direction as to the overall cost to achieve a specific target.

Target Levels for the Reduction of CSO's and WPCP Bypasses/Effluent

Three target levels were established after a review of existing RAP documents, discussions with the Technical Committee and a preliminary assessment of standards used elsewhere. These target levels are a:

- reduction of CSO's to four overflows per year (frequency standard);
- reduction of CSO's to one overflow per year (frequency standard);
- reduction of key water quality parameters to the levels as required in the report entitled "Preferred Options Report for Hamilton Harbour".

The latter generally implies a 90 to 95 percent reduction in constituent loads from CSO's and a 75 to 90 percent reduction in constituent loads from the WPCP.

Target Levels for the Reduction of Constituent Loads from Separated Stormwater Runoff

Data contained within RAP documents, and a review of the options available to reduce constituent loadings from separated storm sewers within existing and future developments were used to determine appropriate target levels for the reduction of constituent loads from separated stormwater runoff. The adopted "targets" are outlined below.

1. Loadings from existing separated storm sewer services are to remain "as is".
2. Loadings from areas slated for development are to be kept as close to existing levels as possible using Best Management Practices.
3. Loadings from remaining rural lands (i.e. farms) are to be reduced in excess of 30 percent in order to meet the loading objectives set forth by RAP.

The latter point will have to be addressed in future studies as the reduction of constituent loadings from non–urban areas is beyond the scope of the study.

Identification and Screening of Remedial Alternatives

An extensive list of remedial options to reduce constituent loads to the receiving water bodies was developed. The remedial options considered were broken down into the following categories:

- source control;
- sewer system control;
- treatment strategies.

Each option was reviewed based on the following factors:

- potential level of improvement;
- technical feasibility;
- cost–effectiveness;
- implementation considerations;
- its ability to meet the desired objectives;
- municipal and public acceptance;
- conflict potential;
- environmental assessment process;
- operation and maintenance considerations.

The review itself was based on published literature, past experiences of the study team and consideration of the specific features of the sewage systems within the Region. Options that were identified as being unfeasible, not cost–effective or inappropriate were removed from further consideration. This process yielded ten recommended control alternatives which were subsequently evaluated to determine their:

- overall effectiveness at reducing constituent loads to the receiving water bodies;
- cost; and
- additional benefits, if any.

Ultimately, both interim and long term control strategies were developed to reduce CSO/WPCP bypass volumes and frequencies. A strategy to reduce constituent loadings from storm water was also

devised for future developments within the Region. An overview of the recommended strategies follows.

Interim Control Strategies

Flow Regulator Adjustment

Flow regulator adjustment was ascertained to be an inexpensive and effective means to reduce CSO frequencies and volumes near human contact areas in the interim. Flow regulator adjustments consist of making the following modifications to direct more combined sewage to the WPCP, rather than to the receiving water bodies:

- raise the height of the overflow weir and/or
- raise the height of the sluice gate.

These adjustments may reduce CSO frequencies by 0 to 40 percent, and CSO volumes by 5 to 60 percent, depending on the site. Outfalls where flow regulator adjustments may be used to reduce CSO frequencies and volumes include King, Dundurn, Hess, James, Catharine and Wentworth.

WPCP Modifications

At the WPCP, interim modifications would allow the plant to handle the following:

- existing dry weather flow
- improved treatment of existing stormwater flow; and
- treatment of flows presently bypassed around the plant.

In terms of storm magnitude, the plant would be able to treat storm flows of the size of Hurricane Hugo (a 25 to 40 mm event), if the regulators in the combined sewers are at their present settings.

Required modifications include:

- improved secondary clarifiers

- application of step feed
- operating the sludge residence time (SRT) at 7 to 8 days
- installing flow control devices and monitoring equipment to control the split in flow between the north side and south side.

Expansion of the Woodward Avenue WPCP will be required, however, to avoid process bypassing during rainfall events if the interim flow regulator adjustments outlined above are made.

Long Term Control Strategies

Long term control strategies to control CSO's consist of remedial works which result in either major or minor reductions in CSO's. Major reductions in CSO's may be attained through the use of either inline or offline CSO storage facilities. These facilities store CSO's, which would normally be discharged to the receiving water bodies, for treatment until the treatment capacity becomes available at a WPCP. Minor reductions in CSO's may be attained through:

- roof downspout disconnection;
- the use of swirl concentrators to treat CSO's;
- public education programs; and
- Real Time Control.

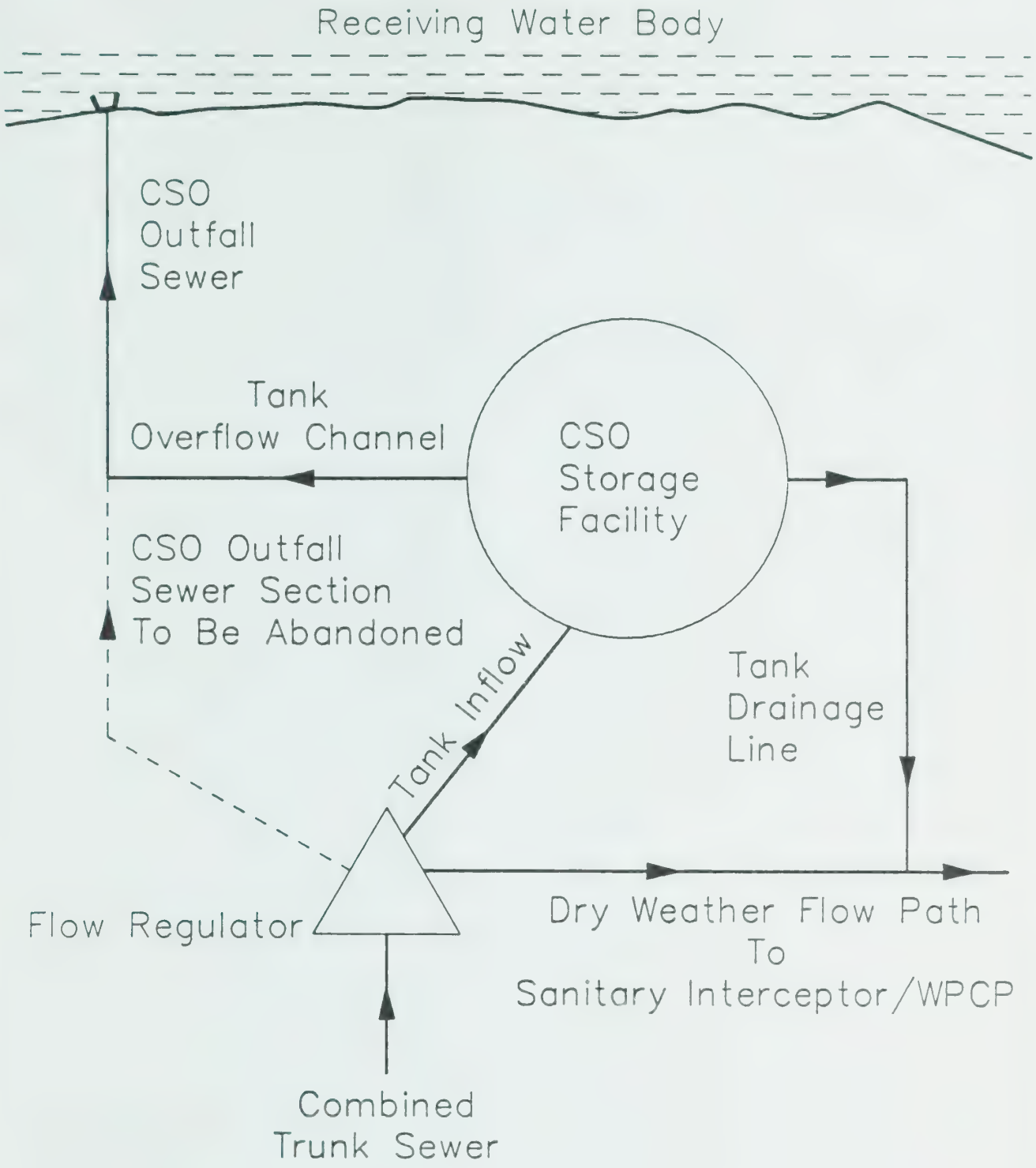
An overview of these recommended remedial works follows.

Offline and Inline Storage of CSO's

Offline, or end of line storage facilities, generally involve the storage of overflows at or near the point of discharge in either above or below ground level facilities (Figure 6). This structural alternative is commonly used throughout North America and Europe. An example of such a facility is the Region's Greenhill Avenue storage facility. Generally, offline storage facilities are an effective and feasible alternative when:

- large facilities may be constructed to provide economies of scale; and
- land exists to locate the facilities.

Figure 6



TYPICAL OFFLINE CSO STORAGE FACILITY

N.T.S.

Applying the latter point to the Hamilton area suggests the use of offline storage facilities may be limited in Hamilton's east end due to high land costs and extensive industrial development. Conversely, Hamilton's west end has more open lands and parks where offline storage facilities may be located.

Inline storage involves the storage of CSO's within the sewer system. Inline storage for larger systems is commonly provided by constructing a tunnel sewer, well below the surface. An advantage of inline over offline storage is that land requirements (and costs) may be kept to a minimum if city streets and existing easements are utilized. Disruption to homeowners, and to traffic, is also minimized if inline facilities are constructed. Consideration of these items suggests inline storage may be an effective remedial option in Hamilton's east end industrial areas where land is not readily available.

Five combinations of inline and/or offline CSO storage facilities were considered in order to quantitatively and qualitatively determine the most effective balance between inline and offline storage. Table 3 summarizes the key parameters and the costs of each alternative. Discussion with the Technical Committee have suggested that either Option 3 (Figure 7) or Option 4 (Figure 8) is the preferred alternative for the reasons outlined below.

- Land is expensive and difficult to acquire, for the offline CSO storage facilities in Hamilton's east end for Options 1 and 2.
- Construction of an inline tunnel from the Woodward Avenue WPCP to Highway 403 (Option 5) would require substantially higher capital costs. Sufficient public lands also exist near Highway 403 for the construction of offline storage facilities.

- The construction of an inline tunnel interceptor could have the added capability, and benefit, of providing an alternate route for combined sewage to be conveyed to the WPCP should the existing western interceptor, which was constructed in the 1950's, be in need of repair.

Long term control strategies within the sewer system which result in minor reductions in CSO's are summarized below.

TABLE 3

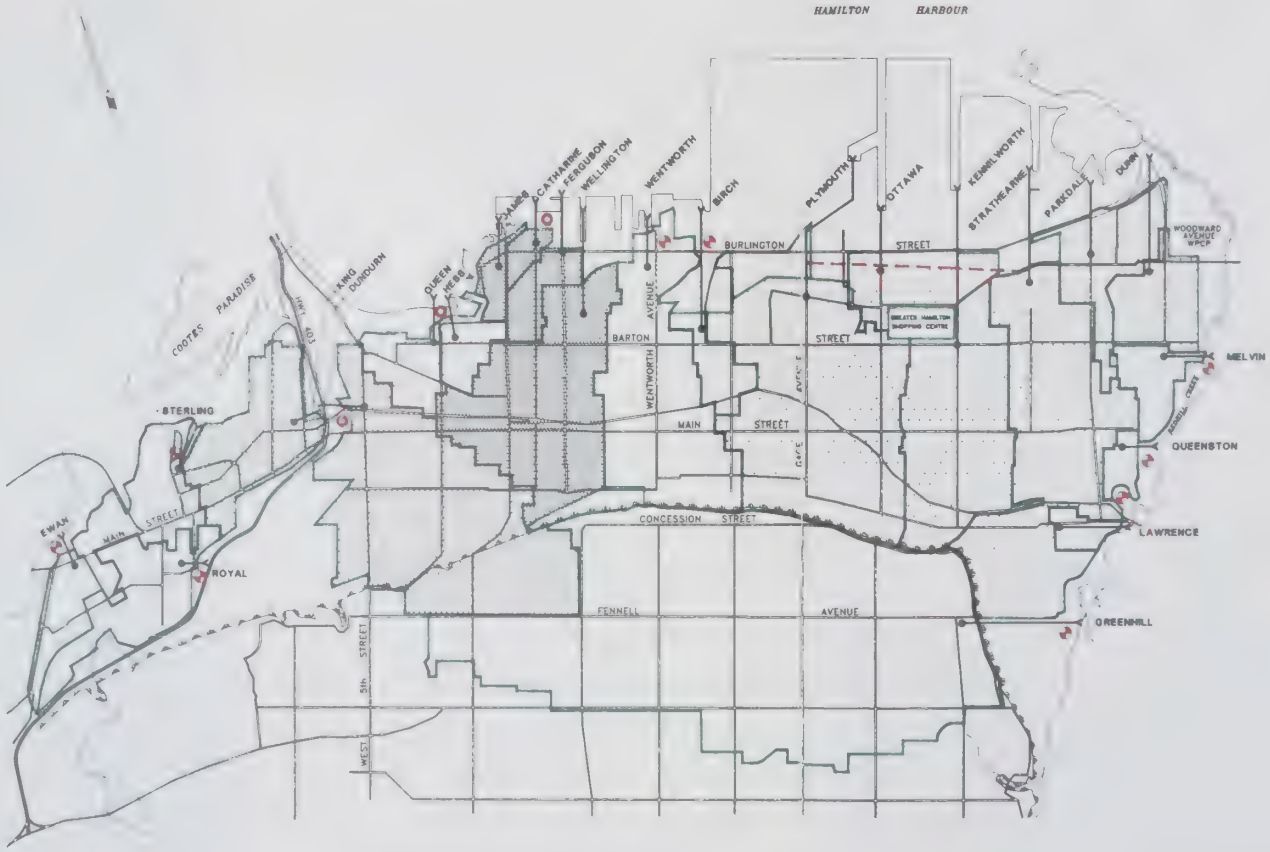
CSO STORAGE ALTERNATIVES AND ASSOCIATED COSTS

Option and Description	Number of Offline Facilities*	Length & Diameter of Inline Tunnel	4 CSO/Year Control Strategy				1 CSO/Year Control Strategy			
			Cons'n Costs		Land Costs	Total Costs **	Cons'n Costs		Land Costs	Total Costs **
			Tanks	Tunnel			Tanks	Tunnel		
1. Separate Offline Storage Facilities at each outfall	22	-	72.1	0	4.0	76.1	115.3	0	5.8	121.1
2. Offline Storage Facilities with some Lumped Facilities	18	-	70.5	0	3.7	74.2	113.8	0	5.2	119.0
3. Offline Storage Facilities and Inline Storage from the Woodward Avenue WPCP to Gage Avenue	12	Length = 4.5 km D=7.0m for 4 CSO's D=9.0m for 1 CSO	42.5	52.0	1.3	95.8	72.3	65.0	1.8	139.1
4. Offline Storage Facilities and Inline Storage from the Woodward Avenue WPCP to Wentworth Avenue	10	Length = 6.2 km D=7.0m for 4 CSO's D=8.5m for 1 CSO	34.2	71.0	0.9	106.1	60.0	86.8	0.8	147.6
5. Offline Storage Facilities and Inline Stroage from the Woodward Avenue WPCP to Highway 403	9	Length = 11.2 km D=6.0m for 4 CSO's D=7.0m for 1 CSO	24.0	112.0	0.3	136.3	44.7	129.0	0.9	174.6

* Excluding the existing Greenhill CSO Storage Facility.
All costs in Million Dollars. Cost estimate data is included in Appendix E.

** Excludes WPCP expansion costs of $90 and $100 million for the 4 and 1 CSO/year control strategies, respectively.

Figure 7



REGIONAL MUNICIPALITY OF HAMILTON - WENTWORTH

POLLUTION CONTROL PLAN

LEGEND

- Combined Sewershed
- Combined Sewersheds Draining To A Common CSO Storage Facility
- CSO Outfall
- Offline Storage Facility
- Lumped Offline Storage Facility
- Proposed CSO Tunnel Interceptor

CSO STORAGE OPTION #3 OFFLINE STORAGE FACILITIES AND INLINE STORAGE FROM THE WOODWARD AVENUE WPCP TO GAGE AVENUE

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Figure 8

REGIONAL MUNICIPALITY OF HAMILTON - WENTWORTH

POLLUTION CONTROL PLAN

LEGEND

- Combined Sewershed
- Combined Sewersheds Draining To A Common CSO Storage Facility
- CSO Outfall
- Offline Storage Facility
- Lumped Offline Storage Facility
- Proposed CSO Tunnel Interceptor

CSO STORAGE OPTION #4 OFFLINE STORAGE FACILITES AND INLINE STORAGE FROM THE WOODWARD AVENUE WPCP TO WENTWORTH AVENUE

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Roof Downspout Disconnection

Roof downspout disconnection reduces CSO's since less water is allowed to enter the combined sewer system if roof runoff is directed to grassed or other areas, rather than to the sewer system. Drawbacks of roof downspout disconnection are:

- homeowners may not be willing to co–operate; and
- past studies, and analysis, have suggested that roof downspout disconnection would decrease CSO storage requirements marginally by approximately 5 percent.

For these reasons, the use of roof downspout disconnection to reduce CSO's may be most feasible in areas where the local sewer system is undersized (e.g. where basement flooding exists).

Swirl Concentrators

Swirl concentrators are compact solids separation devices which are designed to remove heavy organic and inorganic material such as sand and gravel. Floatables are also generally removed. Unfortunately, swirl concentrator designs to date generally only remove 20 to 50 percent of the incoming suspended solids material. For this reason, stand alone swirl concentrators are not an effective means to reduce CSO loadings to receiving water bodies.

It may be beneficial, however, to use swirl concentrators in conjunction with CSO storage facilities to reduce the volume, and hence cost, of the storage facilities. The results of current and future studies on swirl concentrators should be reviewed to determine the feasibility of this option.

Real Time Control (RTC)

A Real Time Control System involves the collection and analysis of data using an automated/computerized system. These systems make prediction of flows and volumes within a sewer system in order to maximize (through the use of gates, pumps, etc.) the available storage and conveyance capacity within a system to reduce CSO's. A RTC system could, therefore, be a feasible alternative for maximizing the use of existing storage facilities or refining the size of the proposed

facilities. This option, however, was not considered in detail as one of the mandates of an existing study is to evaluate the effectiveness of a Real Time Control system.

Public Education Programs

Public education programs involve the preparation and dissemination of information regarding practices which may be undertaken to improve overall water quality. Although public education programs may not result in substantial pollutant loading reduction to the receiving waters, they are generally cost–effective. For this reason, public education programs should be continued or enhanced.

WPCP Remedial Works

Long term expansion options at the WPCP were evaluated for the 1 overflow/year and 4 overflow/year control strategies. The works required at the WPCP are given in the main report. For control of the 1 overflow/year strategy, expansion requirements include:

1. Adding two pumps to the pumphouse to bring it its full complement.

2. Expanding the primary clarifiers by 2.4 times (i.e. add primary clarifiers equivalent to 1.4 times the existing primary clarifiers).

3. Expanding the second clarifiers by 1.6 times (i.e. add new secondary clarifiers equivalent to 0.6 times the existing secondary clarifiers).

4. Expanding the secondary aeration tanks by doubling the south side (i.e. provide additional aeration tanks equivalent in size to the present south size aeration tanks).

5. Adding a new detritor unit in parallel to the existing detritor unit; connect this unit to the existing pumphouse in parallel with the existing detritor unit and then connect it to the new primary tanks.

6. Providing an equalization tank at the WPCP, in conjunction with Storage Options 3 or 4, equivalent to the volume of storm water directed to the plant and not stored in the CSO facilities (160 ML for a one overflow event). The equalization tank is required to control the storm surge as it arrives at the plant.

These works should meet existing C. of A. requirements for the plant, allow operating flexibility, improve nitrification in the effluent, handle anticipated dry weather flows from a population of 500,000 people, and treat normal dry weather flows plus stored CSO volumes from a one overflow event/year control strategy. It is estimated to cost approximately $100 million.

For the 4 CSO/year strategy, the estimated cost is 90 million for WPCP expansion. Expansion requirements are similar to those for the 1 CSO/year strategy, with smaller sizes required for the primary and secondary clarifiers and the equalization tank.

Before implementing these works at the WPCP, some interim improvements in plant performance can be obtained by using step–feed and improving the secondary clarifiers on the north side. However, new construction of primary clarifiers can be anticipated at an early date, as a minimum requirement. Overall, substantial retrofitting will be required at the WPCP to implement these works.

To assist in evaluating these interim improvements and to check suggested locations for the process units in the expanded plant, a planning process is required. Elements of the plant planning process are described in the main report (Section 9). They include:

1. Evaluation plant improvements possible from the ongoing studies (1991).

2. Based upon additional calibration data, and refined objectives for plant performance, examine the simulation model calibration and adjust plant expansion requirements into several time frames, for example:
   0 – 5 years
   5 – 10 years
   10 – 15 years
   15 – 20 years
   20 – 25 years

Expansion requirements should take into account the peaking of the influent hydrograph until system storage units or equalization tanks are built (this could be done in 1992 to 1993.

3. Initiate detailed studies of other treatment options (1992).

4. Develop a plan for plant improvements to the present plant.

5. Develop a detailed plan for plant expansion. We see this to involve the following elements:
   - add additional primary clarifiers fairly immediately, since the present clarifiers are already overloaded
   - evaluate the performance of additional clarifiers and determine whether a biological treatment strategy for storm flows is the preferred option, or whether physical–chemical options should be used
   - determine whether additional aeration tanks and final clarifiers should be built, and if so, their size.

Remedial Works for Separated Stormwater Systems

The recommended control strategy to reduce constituent loadings from separated stormwater systems in future developments is to use stormwater management (wet) ponds, or equivalent technology, to treat the runoff. Wet ponds are passive devices which remove contaminants from stormwater runoff by settling solids and chemicals (e.g. heavy metals) attached to the solids. Wet ponds are expected to remove approximately 80 percent of SS, 50 percent of TP and substantial quantities of most other constituents as well. Wet ponds may also be incorporated into social (i.e. recreational) environments to provide additional benefits to the public. Capital costs for wet ponds are also expected to be a minimum, since costs are expected to be shared by land development companies developing within an area. Wet ponds normally require a surface area of less than 5 percent of the upstream drainage area for effective operation, and a minimum drainage area of approximately 10 ha. No wet ponds currently exist within the Region, nor are new developments required to install them.

Reduction in Constituent Loadings with Proposed Works

The reduction in constituent loadings to the receiving water bodies for a typical (1989) year was ascertained with:

- CSO control strategies of 1 and 4 CSO events/year;
- CSO storage alternatives 3 or 4;
- the proposed WPCP works to maintain current effluent quality; and
- stormwater management (wet) ponds in all future developments.

The analysis was conducted for both existing and future conditions.

For comparison purposes, all loadings were measured according to their percent increase or decrease relative to existing conditions. Conclusions which may be drawn from the analysis are outlined below.

1. CSO loadings reduced by 95 to 87 percent with the 1 and 4 CSO/year control strategies, respectively. CSO loadings are not expected to increase under future conditions since the combined sewer drainage area is essentially fully developed.

2. Under existing conditions, WPCP inflow volumes increase by approximately 6 percent under either CSO control strategy since more sewage is directed to the WPCP by the CSO storage facilities. Accordingly, since the WPCP remedial works are designed to maintain, rather than enhance effluent quality, WPCP loadings also increase (by approximately 3 percent). WPCP loadings do not increase to the same extent as WPCP inflow volumes since the additional sewage is more dilute than normal WPCP inflows.

3. Under future conditions, loadings from the WPCP are expected to increase by approximately 33 percent. Approximately 90 percent of this increase may be attributed to the expected increase in population in the sewershed. The remaining increase originates from the diversion of CSO flows to the WPCP by the CSO storage facilities.

4. Under future development conditions, loadings from separated stormwater systems are expected to increase 40 percent without stormwater management (wet) ponds, and 15 percent with wet ponds.

5. Under existing development conditions total constituent loadings are expected to decrease approximately 15 percent with either CSO control strategy (Table 4). Generally, the greater the contribution of CSO's to the existing loading of a constituent, the greater the reduction in the total loading.

6. Generally, if CSO's contribute more than 15 percent of the existing loadings, then loadings under future conditions are expected to be approximately 3 to 35 percent less with the recommended CSO, WPCP and stormwater remedial works. This trend is illustrated for SS, Zn, FC and Pb loadings in Table 4. Conversely, if CSO's currently contribute less than 15 percent of the existing loadings, then loadings under future conditions are expected to be approximately 10 to 30 percent greater. This trend is illustrated for TS, TP, NH_3, TKN, BOD_5 and Cu loadings (Table 4). The loadings in the latter case increase since the WPCP contributes the majority of the loadings and WPCP inflow volumes increase due to increased growth in the area.

7. Constituent loadings to Hamilton Harbour/Cootes Paradise under future conditions with the proposed remedial works are highly variable, ranging from a decrease in loadings of 42 percent, to an increase of approximately 29 percent (Table 4). Overall, the general trend is for loadings to increase by approximately 6 percent, compared with a 28 percent increase if no remedial works are constructed. Further loading reductions could be attained by improving the quality of the Woodward Avenue WPCP effluent.

8. The long term changes in SS, TP and TKN loadings, which are pollutants of particular concern to RAP, are as illustrated in Figure 9.

As outlined earlier, recent RAP documents have suggested that approximately a 90 percent reduction in constituent loadings from CSO's and the WPCP is required in order to obtain the desired water quality in the receiving waters. This point suggests a 1 CSO/year control strategy is required for CSO's, in addition to WPCP effluent polishing to lower constituent concentrations in the WPCP effluent. Furthermore, as the ultimate goal of the RAP to lower constituent loadings to below existing

TABLE 4

CONSTITUENT LOADINGS TO HAMILTON HARBOUR

Constituent	Loading For Existing Conditons (kg/day)	Percent of Existing Loading Contributed by CSO's	Percent Increase (+) or Decrease (-) in Total CSO, WPCP and Stormwater Loadings for Stated Conditions				
			Existing Level of Dev. With Recommended CSO and WPCP Remedial Works		Future Conditions With No Remedial Works*	Future Conditions With Recommended CSO, WPCP & Stormwater Remedial Works	
			4 CSO/Yr Control Strategy	1 CSO/Yr Control Strategy		4 CSO/Yr Control Strategy	1 CSO/Yr Control Strategy
SS	15,700	17	-14	-15	+27	-3	-4
TS	186,000	5	-2	-2	+29	+26	+26
TP	177	9	-8	-9	+28	+18	+18
NH_3	2,140	1	-1	-1	+29	+29	+28
TKN	3,220	2	-1	-2	+29	+27	+27
Zn	20	40	-35	-35	+25	-20	-20
FC (counts/day	$3.6x10^{13}$	60	-50	-56	+22	-36	-42
BOD_5	3,700	11	-10	-10	+29	+19	+18
Pb	13	25	-23	-23	+31	-8	-15
Cu	9	10	-11	-11	+33	+11	+11
Average	-	18	-15	-16	+28	+7	+5

* Assumes Woodward Avenue WPCP is expanded to provide additional treatment capacity for population increases only.

SS,TP and TKN LOADINGS TO HAMILTON HARBOUR / COOTES PARADISE

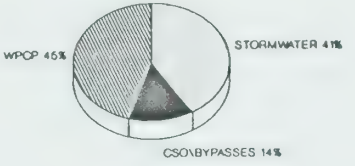
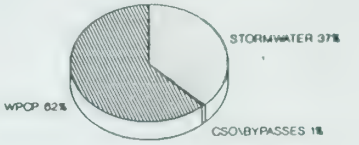
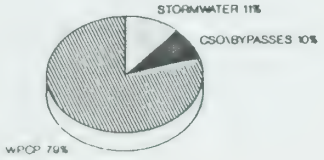
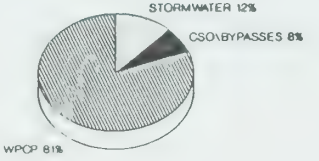
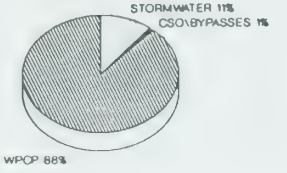
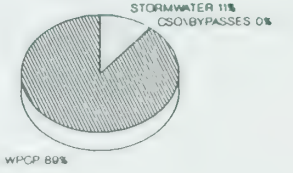
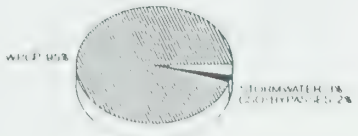
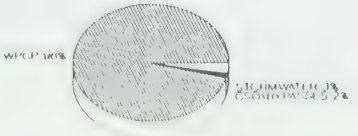
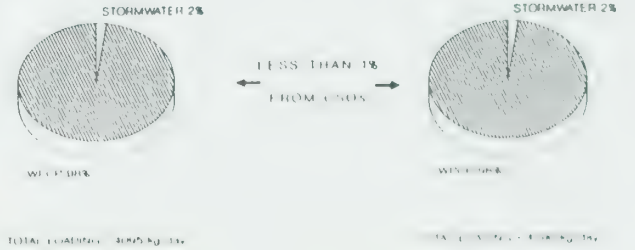
	EXISTING CONDITIONS	FUTURE CONDITIONS NO REMEDIAL WORKS		FUTURE CONDITIONS 4 CSO/YEAR CONTROL STRATEGY & STORMWATER RUNOFF CONTROLS	FUTURE CONDITIONS 1 CSO/YEAR CONTROL STRATEGY & STORMWATER RUNOFF CONTROLS
SUSPENDED SOLIDS (SS)	TOTAL LOADING = 16,000 kg/day	 TOTAL LOADING = 20,000 kg/day		TOTAL LOADING = 15,200 kg/day	 TOTAL LOADING = 15,000 kg/day
TOTAL PHOSPHORUS (TP)	 TOTAL LOADING = 177 kg/day	 TOTAL LOADING = 226 kg/day		 TOTAL LOADING = 207 kg/day	 TOTAL LOADING = 208 kg/day
TOTAL KJELDAHL NITROGEN (TKN)	 TOTAL LOADING = 12,500 kg/day	 TOTAL LOADING = 4,850 kg/day		 TOTAL LOADING = 4,045 kg/day	

Figure 9

levels, steps will have to be taken to improve stormwater runoff quality from rural areas (e.g. farms) within the Hamilton Harbour/Cootes Paradise drainage basin.

Estimated Total Cost of Proposed Works

Total estimated capital costs of approximately $190 to $250 million have been ascertained to:

- reduce CSO's to either 4 or 1 CSO events per year, respectively; and to
- treat the stored CSO's at the Woodward Avenue WPCP while maintaining current effluent quality.

Breakdowns of these costs are provided in Table 5.

These capital costs are equivalent to annual expenditures of approximately $10 to $12.5 million if a 20 year implementation plan is adopted. Costs for stormwater management (wet) ponds are expected to be borne by land development companies.

Conclusions

Outlined below are the key conclusions which may be made based on the findings in this study.

1. CSO's occur once every eight days or so through the Region. During a typical May 1 to October 31 period approximately 4.4 million cubic metres of combined sewage will be discharged to the receiving waters. CSO's contribute less than 20 percent of the total loadings to Hamilton Harbour under existing and future conditions.

2. Inexpensive interim remedial works may be used to reduce CSO frequencies by up to 40 percent and CSO volumes by up to 60 percent, depending on the site. Six CSO outfalls by human contact areas were identified where interim works may be applied.

3. CSO frequencies and volumes may be reduced substantially by constructing inline and offline CSO storage facilities. By adopting a 1 or 4 CSO/year control strategy, CSO volumes and loadings are expected to decrease by 95 and 87 percent, respectively. In

TABLE 5

TOTAL COST OF RECOMMENDED REMEDIAL WORKS

Level of Control	CSO Storage Option	Cost of CSO Storage (million)	WPCP Improvement Costs (million)	Estimated Total Cost (million)
4 CSO's/year	3. Offline storage facilities and inline storage from the Woodward Avenue WPCP to Gage Avenue (Figure 7).	95.8	90	186
4 CSO's/year	4. Offline storage facilities and inline storage from the Woodward Avenue WPCP to Wentworth Avenue (Figure 8).	106.1	90	196
1 CSO/year	3. Offline storage facilities and inline storage from the Woodward Avenue WPCP to Gage Avenue (Figure 7).	139.1	100	239
1 CSO/year	4. Offline storage facilities and inline storage from the Woodward Avenue WPCP to Wentworth Avenue (Figure 8).	147.6	100	248

order to meet RAP loading objectives for CSO's, it is expected that a 1 CSO/year control must be adopted. Capital costs for the construction of the required facilities are estimated to be approximately $145 million (1990 dollars).

4. In general, under future development conditions, and with the treatment of stored CSO flows at the WPCP, the capacity of the Woodward Avenue WPCP will have to double if current effluent quality is to be maintained. Capital costs for this level of expansion are estimated to be $100 million (1990 dollars).

5. Stormwater management (wet) ponds are appropriate to provide stormwater quality control for new developments.

6. The Woodward Avenue WPCP is the largest contributor of pollutants to Hamilton Harbour/Cootes Paradise. Therefore, substantial reductions in constituent loadings to Hamilton Harbour will require the improvement of WPCP effluent quality.

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REGIONAL MUNICIPALITY OF HAMILTON–WENTWORTH POLLUTION CONTROL PLAN

1 – INTRODUCTION

1.1 General

The Regional Municipality of Hamilton–Wentworth and the Ontario Ministry of the Environment initiated the Regional Municipality of Hamilton–Wentworth Pollution Control Plan (RMHWPCP) to develop a strategy for reducing pollutant loads to the local receiving water bodies. Conclusions and recommendations from the RMHWPCP may be used to assist in the development of a Remedial Action Plan (RAP) which, when implemented, will improve the water quality in the local receiving waters. The RAP will be a joint effort between the Region, Federal and Provincial agencies and stakeholders.

The study area for the RMHWPCP is the Regional Municipality of Hamilton–Wentworth, with the core study area being the City of Hamilton. Within the study area, four major receiving water bodies exist, these are:

- Hamilton Harbour;
- Chedoke Creek;
- Cootes Paradise; and
- Redhill Creek.

Chedoke Creek, Cootes Paradise and Redhill Creek discharge to Hamilton Harbour, while Hamilton Harbour discharges to Lake Ontario.

Sources of contaminated discharges to the receiving water bodies are:

- combined sewer overflows (CSO's);
- sewage treatment plant (WPCP) bypasses and effluent;
- separated stormwater system discharges.

Each of these sources were considered in the study, although CSO's and WPCP discharges were the main items of concern.

1.2 Study Objectives

The objectives of this study are outlined below.

1. Ascertain the constituent loads to receiving water bodies from CSO's, WPCP discharges, and separated stormwater discharges. The constituents of interest, as specified by the Terms of Reference, are:

   - fecal coliform (FC)
   - total solids (TS)
   - suspended solids (SS)
   - 5-day biochemical oxygen demand (BOD_5)
   - total phosphorus (TP)
   - ammonia (NH_3)
   - lead (Pb)
   - copper (Cu)
   - Zinc (Zn)
   - total kjeldahl nitrogen (TKN).

2. Based on the loading results, prioritize the pollutant sources.

3. Develop and evaluate remedial strategies such that a range of pollutant load reductions may be attained. Emphasis will be placed on CSO's. The remedial measures will consider both existing and ultimate development conditions.

4. Evaluate the cost-effectiveness, feasibility and implementation considerations of the proposed remedial measures.

1.3 Study Methodology

The study team completed the RMHWPCP study in a logical and systematic manner. The key tasks completed to develop the RMHWPCP are outlined below.

1. Existing sources of information and data were reviewed to develop an understanding of the various systems under study. The information reviewed included:

   - past reports completed for the Region;
   - experiences of Region staff;
   - WPCP records;
   - Atmospheric Environment Service (AES) records;
   - appropriate technical papers.

2. Data acquisition programs were initiated to acquire additional information on sewershed conditions and WPCP processes.

   The programs initiated included:

   - flow and rainfall monitoring;
   - water quality sampling;
   - the logging of WPCP bypassing and WPCP controlled sluice gate closures;
   - field investigation to ascertain critical hydrologic and hydraulic characteristics of the study area;
   - meetings with Region staff.

3. Appropriate hydrologic/hydraulic and WPCP process models were developed for analysis of the various systems.

4. Constituent loadings to the receiving water bodies were ascertained.

5. Remedial works to reduce constituent loadings to the receiving water bodies were proposed and assessed.

6. Conclusions and recommendations were made based on the findings in the study.

2 - DESCRIPTION OF THE STUDY AREA

2.1 General

All lands within the Regional Municipality of Hamilton-Wentworth are included in the study area. Included within this regional boundary are the municipalities of Hamilton, Stoney Creek, Dundas, Ancaster, Glanbrook and Flamborough. The core study area for the study is the City of Hamilton, where numerous combined sewer overflows and the main sewage treatment plant exists. A population of approximately 430,000 people resided in the study area in 1988, of which 310,000 people resided within the City of Hamilton.

2.2 Municipal Sewage System

The sewage system within Hamilton-Wentworth includes both combined and separated sewer systems, and three sewage treatment plants.

Within the core area study area three drainage basins exist (Figure 2.1). The Chedoke Creek basin covers the western portion of Hamilton, contains predominantly residential housing, and is approximately 50 percent serviced by combined sewers. CSO's from the Chedoke Creek basin are discharged to Chedoke Creek and to Cootes Paradise. The Central Business District basin covers Hamilton's industrial and commercial core and is essentially 100 percent serviced by combined sewers. CSO's from the Central Business District are discharged directly to Hamilton Harbour. Combined sewers service approximately 25 percent of the Redhill Creek basin. Residential and commercial lands dominate the northern portions of the Redhill Creek sewershed while rural/agricultural lands exist in the southern areas. CSO's from the Redhill Creek basin are discharged directly to Redhill Creek.

Outside of the core study area there are an additional five drainage basins which discharge stormwater flows to Hamilton Harbour/Cootes Paradise, namely: Grindstone Creek, Spencer Creek, Rock Chapel Creek, Sulpher/Ancaster Creek and the industrial areas along the waterfront (Figure 2.2).

Most future development within this area is expected to occur within the Redhill and Ancaster/Sulpher Creek watersheds. Table 2.1 and Figure 2.3 summarize the existing and estimated future conditions

Figure 2-1

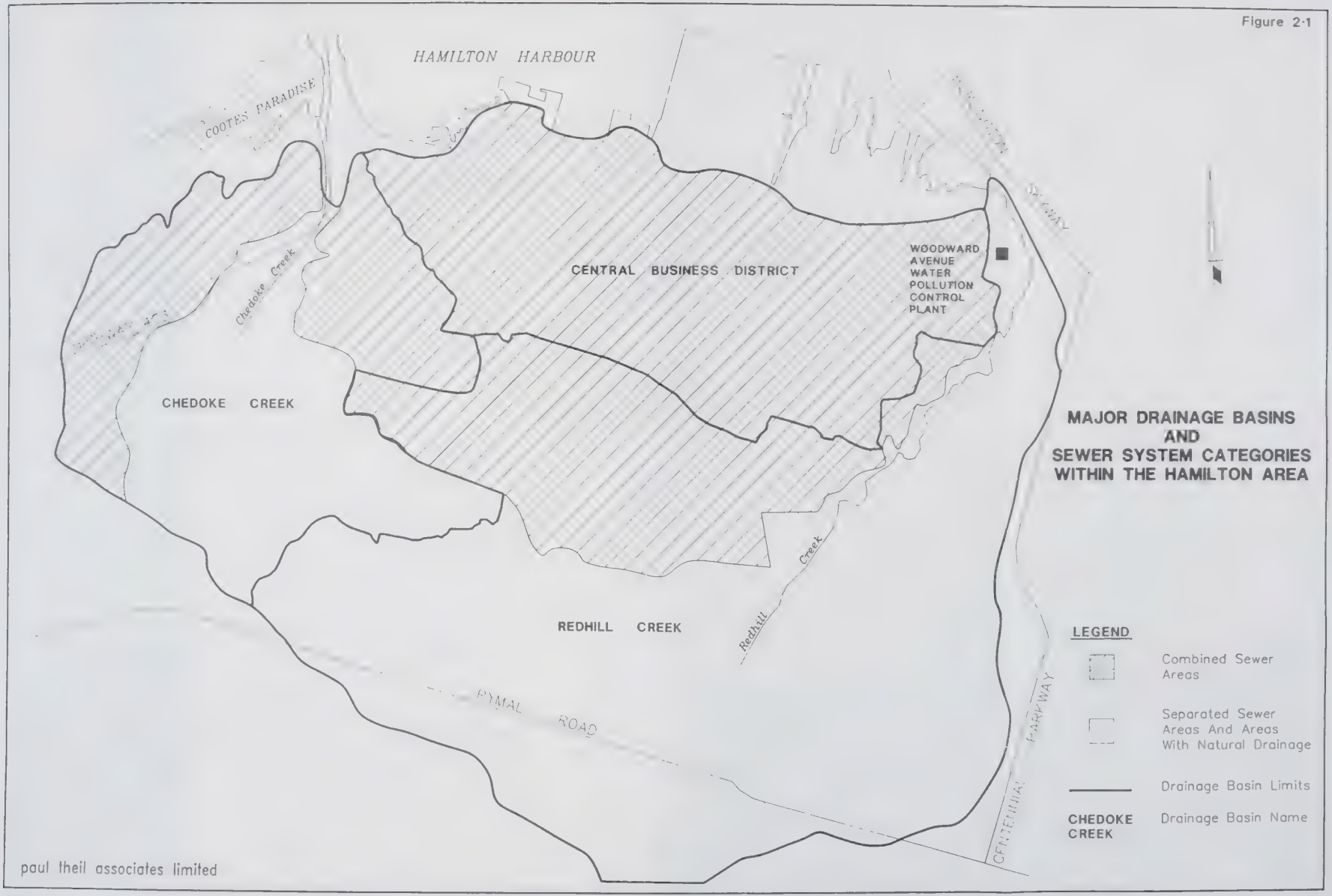

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Figure 2·2

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Figure 2·3

REGIONAL MUNICIPALITY OF HAMILTON - WENTWORTH

EXISTING AND PROPOSED LAND USES

LEGEND

- Existing Development
- Proposed Industrial–Business Parks
- Proposed Residential and Related Uses
- Combined Sewer Area
- Parkway Belt West Policy Areas
- Rural Area
- Rural Settlements
- Niagara Escarpment
- Airport
- Regional Boundaries

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Source: The Regional Development Pattern (1989)

in the area and the corresponding breakdown of drainage areas discharging to Hamilton Harbour/Cootes Paradise and indicates:

- under existing conditions combined and separated storm sewer areas each service approximately 50 percent of the developed lands;

- with separated sewers installed in proposed development areas, approximately two–thirds of the developed areas will be serviced by separated storm sewers and one–third of the developed land will continue to be serviced by combined sewers.

TABLE 2.1

DRAINAGE AREAS DISCHARGING TO

HAMILTON HARBOUR/COOTES PARADISE

Conditions and Drainage System	Drainage Area (km^2)	Percent of Total Area
Existing Conditions		
Combined Sewer Areas	54	47
Separated Storm Sewer Areas	62	53
TOTAL	116	100
Proposed Development Conditions As Per The Regional Development Pattern (1989)		
Combined Sewer Areas	54	34
Separated Storm Sewer Areas	103	66
TOTAL	157	100

2.2.1 Combined Sewer Systems

Combined sewers provide sanitary and stormwater drainage for most areas within the City of Hamilton. While some of the sewers date back to the 1800's, the majority of the system was constructed between 1900 and 1950. Figure 2.1 illustrates the boundaries of the combined sewer system within Hamilton. This area covers approximately 54 km^2 and services a population of approximately 250,000.

The main components of the combined sewer system are the combined trunk sewers, overflow regulators, sanitary interceptor sewers and CSO outfall sewers. Each of these components are reviewed in Sections 3 and 4.

2.2.2 Separated Sewer System

Separated sewers began to be constructed in Hamilton in the 1960's and are most common in the southern portions of the Redhill Creek basin (i.e. south of Mohawk Road). Sanitary flows from these lands drain to the Redhill Creek sanitary interceptor, while stormwater flows generally drain to the Redhill Creek. Stoney Creek is also serviced by a separate sewer system. Stoney Creek sanitary sewer flows drain to the Redhill Creek sanitary sewer, while storm sewer flows are either discharged to Redhill Creek, Hamilton Harbour, or Lake Ontario. Dundas is serviced by a separate sanitary sewer system which conveys sewage to the Dundas WPCP.

2.3 Water Pollution Control Plants

Three sewage treatment plants service the area: the Woodward Avenue WPCP, the Dundas WPCP and the Waterdown WPCP. The Woodward Avenue WPCP was constructed in the 1950's and provides sewage treatment for Hamilton, Stoney Creek, Ancaster and Glanbrook. The Dundas WPCP and Waterdown WPCP service their respective communities. Table 2.2 summarizes the major components and capacities of each plant.

TABLE 2.2

SUMMARY OF HAMILTON WENTWORTH WPCP'S

PLANT	TREATMENT PROCESSES	POPULATION SERVED	AVG. DAILY FLOW (MLD)	Design Capacities	
				PRIMARY (MLD)	SECONDARY (MLD)
Woodward Ave. WPCP	– Conventional Activated Sludge – Phosphorus Removal By Chemical Precipitation	300,000	303	409	615 – Short Term 545 – Long Term
Dundas WPCP	– Conventional Activated Sludge – Phosphorus Removal By Chemical Precipitation – Continuous – Effluent Polishing with Rapid Sand Filters	19,500	18	–	–
Waterdown WPCP	– Conventional Activated Sludge – Phosphorus Removal – Effluent Polishing With Rapid Sand Filters	–	3	–	–

Note: MLD – million litres per day.

3 – STUDY METHODOLOGY AND DESCRIPTION OF SELECTED SYSTEM COMPONENTS

3.1 Study Methodology

3.1.1 General

Before any detailed system analysis was performed, the study team reviewed the Terms of Reference and met with the Region and the Ministry of the Environment to clarify the direction of the study. This process generated the points outlined below.

1. More time and effort should be spent addressing CSO's in the Region than separated storm sewer discharges.

2. The impact of treating combined sewer overflows at the Woodward Avenue WPCP needs to be ascertained. In particular, what are the process requirements at the WPCP in order to meet the loading reductions as defined by RAP? This assessment should be conducted by determining the impact of CSO treatment and the associated costs on the existing plant. Emphasis was also placed on maintaining current effluent quality at the WPCP.

Having defined the general direction of the study the next step was to determine a systematic procedure for analysis of the combined sewer system, the WPCP, and the storm sewer system. The adopted procedure is outlined below.

1. Review existing plans and reports to develop a "feel" for systems being studied (Section 3).

2. Existing data, and data acquired through the study, should be reviewed for trends. If necessary, initiate data acquisition programs to collect any required data (Section 4).

3. Determine the required level of analysis/modelling for the systems being studied and develop representative models (Section 5).

4. Analyze the systems being studied using the modelling and/or analysis tools developed in Step 3. Each system should be analyzed under a range of conditions to best understand system response (Section 6).

5. Propose and ascertain the effectiveness of any remedial works developed to reduce constituent loadings to the receiving water bodies (Sections 7, 8 and 9).

6. Make final conclusions and recommendations (Section 10).

3.1.2 General Methodology for WPCP Analysis

Based upon the study objectives and the study terms of reference, the major emphasis of this component of the study was to assess the present treatment capability of the Woodward Avenue WPCP during wet and dry weather periods, the impact of CSO diversion on the Woodward Avenue WPCP and possible remedial measures for handling these impacts.

The assessment of treatment plant capacity was based upon:

i) practical operating information which indicate the hydraulic loadings that the plant can accept;

ii) analysis of historical records to derive relationships between rainfall volumes and the impact on the plant;

iii) data gathered in 1989 on the plant where process upsets occurred during storm impacts, and a modelling study of these events;

iv) a model simulation of alternative operating strategies to handle increased hydraulic loadings resulting from storms; and

v) improvements in treatment performance which are indicated by data gathered by this study team during storm events.

The study approach used was the following:

Combined Trunk Sewers

Combined trunk sewers within the Region generally have a north/south orientation and are located under major streets (e.g. Wentworth, Strathearne). These sewers, which generally range in size from approximately 4' X 5' to 12' X 12', collect and route sewage from the local (i.e. 250 mm to 900 mm) combined sewers to the flow regulators which distribute flows between the sanitary interceptors and the receiving water bodies.

Flow Regulators

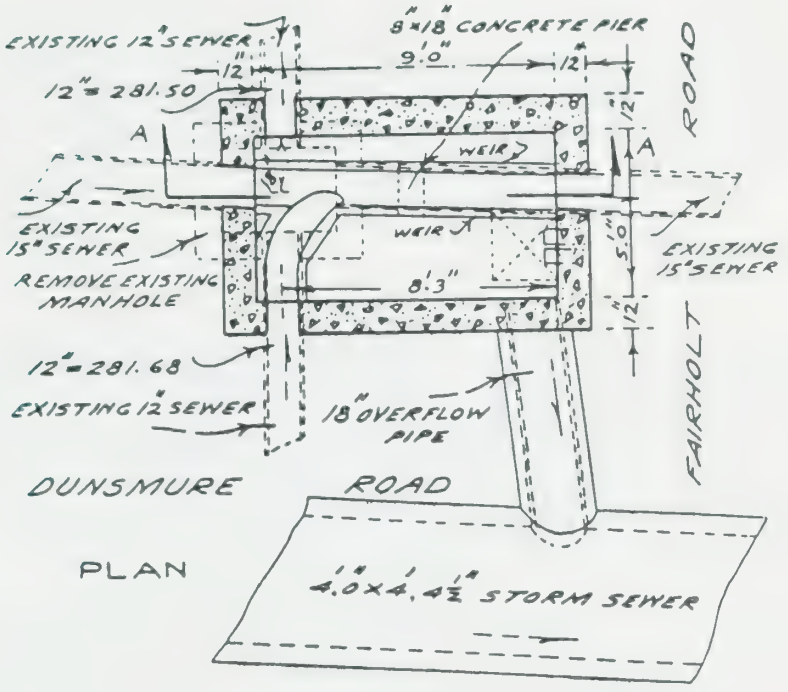
Flow regulators are hydraulic structures which determine the direction, or directions, which an incoming flow will take. For the purpose of this study flow regulators were classified as either major or minor. Generally, minor flow regulators are located on local combined sewers and do not determine sewage discharges to the sanitary interceptor sewers. Rather, the minor flow regulators were constructed in conjunction with the storm relief sewers (i.e. the CSO outfall sewers) to minimize sewer surcharging, and basement flooding on local streets. More recently some local streets have been completely separated to minimize the potential for basement flooding. In excess of 100 minor flow regulators exist within the Region, and most consist of a side–spillway overflow weir (Figure 3.1). Conversely, major flow regulators are located on the combined trunk sewers and generally determine sewage inflows to the sanitary interceptors, and the quantity of CSO's. These regulators consist of weirs, orifice plates, and either fixed or WPCP controlled sluice gates. Approximately 30 major flow regulators exist within the Region. Figure 3.2 illustrates the layout of a typical major flow regulator.

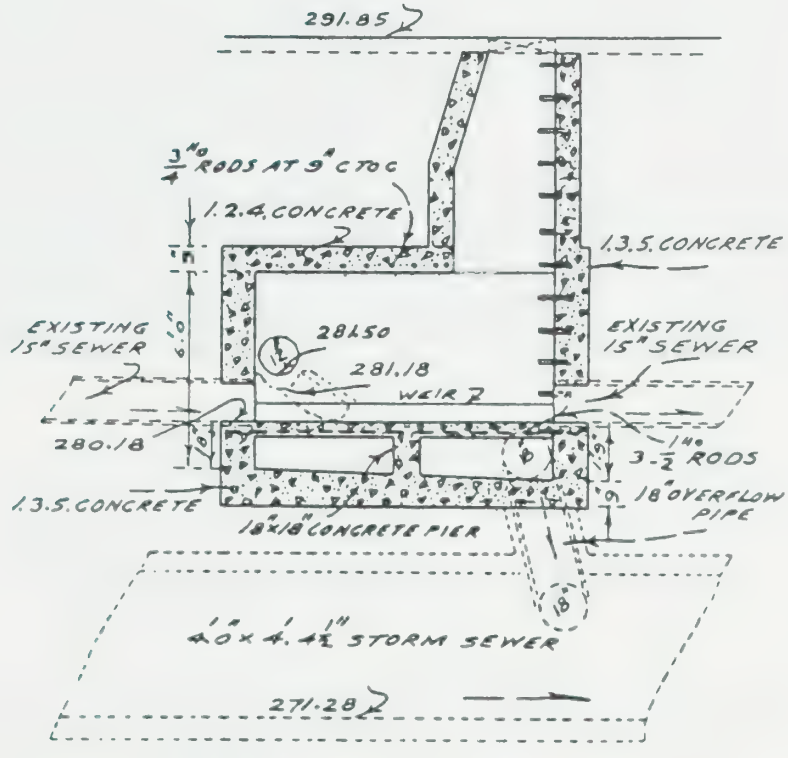
Sanitary Interceptor Sewers

Two main sanitary interceptor sewers service the Region. The western interceptor collects sewage from West Hamilton and the Central Business District, while the Redhill Creek interceptor collects sewage from Stoney Creek and the Redhill Creek sewershed. Both interceptors range from approximately 60" (1500 mm) to 102" (2550 mm) in diameter and convey sewage to the Woodward Avenue WPCP.

The western interceptor was constructed (i.e. tunnelled) in the late 1950's at depths below the surface ranging from approximately 10 m to 25 m. Most of the sewer is situated below Burlington Street and Barton Avenue. This interceptor receives combined sewage from approximately 30 major flow regulators. Although the inflows from these regulators are sufficient to surcharge the interceptor

TYPICAL CSO FLOW REGULATOR ON A LOCAL STREET





TYPICAL MAJOR CSO FLOW REGULATOR

WPCP - CONTROLLED SLUICE GATE
(May be used to stop inflow to Sanitary Sewer)

INCOMING COMBINED SEWAGE FLOW

OVERFLOW WEIR

(Location: East End of Glen Road by Chedoke Creek)

frequently, it is unlikely that the interceptor backs up and overflows to the CSO outfalls due to its depth below the surface.

The Redhill Creek interceptor sewer is situated within the Redhill Creek Valley, was constructed in the early 1960's, and generally lies 5 m to 10 m below the surface. It differs from the western interceptor in that it has fewer (i.e. two) combined sewer inflow points and numerous sanitary sewer inflows. The largest sanitary sewage inflows originate from the City of Stoney Creek and from the separated sewer areas located in the southern portions of the Redhill Creek basin. The largest combined sewer inflow point is located at the Greenhill CSO storage facility.

Greenhill CSO Storage Facility

The Greenhill CSO storage facility, which was constructed in the early 1980's, provides CSO storage for the majority of the Redhill Creek basin. The facility is located east of Greenhill Avenue, west of Redhill Creek, and is approximately 10 m in depth and 100 m in diameter. This facility has a total storage capacity of approximately 80,000 m^3, which is divided between a small cell (13,000 m^3) and a large cell (67,000 m^3). This capacity is generally sufficient to prevent CSO's to Redhill Creek for rainfall events up to approximately 15 mm in depth.

Although the drainage for the facility (i.e. the rate of discharge to the Redhill Creek sanitary interceptor) was designed to be controlled by an automated sluice gate system, Woodward Avenue WPCP staff have operated the facility manually since approximately 1988. WPCP staff switched to manual operation of the gates because the automatic gate system tended to discharge large volumes of combined sewage to the Redhill Creek sanitary interceptor at times when no treatment capacity was available at the WPCP. By operating the gates manually WPCP personnel are able to drain the facility when the capacity exists at the WPCP to treat the sewage. Briefly, manual operation of the gates implies:

1. The gate draining the smaller cell (Cell 1) is constantly open, exposing approximately 20 percent of the 48 inch (1200 mm) drainage pipe which conveys flows to the Redhill Creek Sanitary Interceptor. This gate positioning allows dry weather flows to pass through the facility, but restricts wet weather flows, initiating storage in Cell 1. If Cell 1 fills, additional flows are stored in Cell 2.

2. The gate draining Cell 2 is closed. If sewage is stored in Cell 2 after a rainfall event, then WPCP personnel will drain the facility once the capacity exists at the WPCP to treat the sewage.

3.2.2 Water Pollution Control Plants

Location of Plants

There are three wastewater treatment plants within the Region of Hamilton–Wentworth draining to Hamilton Harbour.

- Woodward Avenue WPCP
- Dundas WPCP and
- Waterdown WPCP

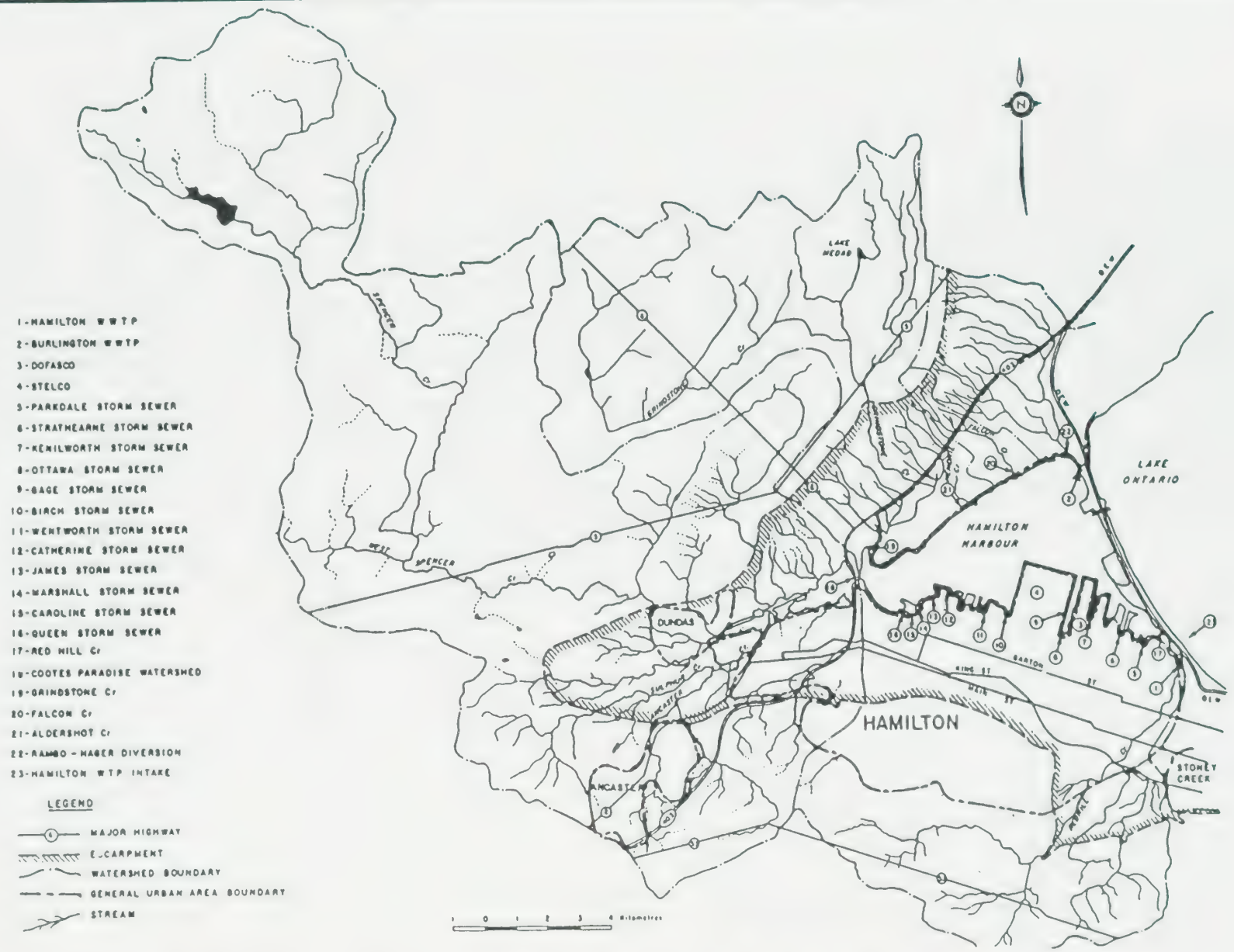
The Hamilton–Wentworth WPCP discharges treated effluent to the south–east corner of Hamilton Harbour (see location 1, Figure 3.3) after receiving sewage from the collection system. The water supplied to the City of Hamilton and the urban areas of Ancaster and Dundas is pumped from Lake Ontario by the Hamilton Water Treatment Plant (location 23, Figure 3.3), prior to use by residences, and industry and returned to the sanitary sewer system.

The Dundas WPCP discharges to Cootes Paradise upstream of the east Pond while the Waterdown WPCP discharges to Grindstone Creek where it flows over the Escarpment (see where Grindstone Creek intersects the Escarpment on Figure 3.3).

The dominant source of loadings of contaminants to Hamilton Harbour is the Woodward Avenue WPCP, due to its high flow rate (approximately 315 MLD; 70 MIGD). The Dundas WPCP discharges a much smaller volume (18 MLD; 4 MIGD) while the Waterdown plant treats an even smaller volume (3 MLD; 0.6 MIGD).

WPCPs Examined in This Study

The terms of reference for the Hamilton–Wentworth Pollution Control Plan Study gave attention to two items:



HAMILTON HARBOUR WATERSHEDS

Figure 3·3

- the impact of the respective treatment plants upon the respective receiving water bodies
- the impact of storm water runoff through the combined sewer system upon the wastewater treatment plants.

The following points result from a review of these items:

1. The Woodward Avenue WPCP receives much of its sewage from a combined sewer system which is located both below the "mountain" as well as on the "mountain".

2. The loading of many substances to Hamilton Harbour together with the harbour's hydrology is dominated by inputs from the WPCP.

3. There is much potential for improvement in Hamilton Harbour water quality by achieving nitrification and improving phosphorus removal in the plant, modifying the sewer system and plant to provide better treatment for storm related surges, and instituting source control through the MISA program to reduce loadings of metals to the plant.

4. The Dundas WPCP has a significant impact upon Cootes Paradise, but a much smaller effect upon Hamilton Harbour water quality.

5. The Dundas WPCP is impacted by storm related surges, but has a much smaller portion of its total sanitary sewer system as a combined system in comparison to the Hamilton WPCP.

6. The Dundas WPCP has extensive tertiary treatment capabilities, achieving nitrification by operating at a suitable sludge age, and enhanced phosphorus removal by rapid sand filtration of the effluent.

7. Substantial questions about whether to continue to operate the Dundas WPCP or to close it and pump the sewage to Woodward Avenue have been raised in the past decade, based upon the additional flow to Woodward Avenue plant being small.

8. A full scale plant experiment using the "step-feed" operating policy to handle storm flows has been successfully demonstrated on the Dundas Plant during the time that this PCP study was being undertaken. This has proven that the Dundas facility could treat storm

flows at approximately 1.5 to 2 times its rated capacity, resulting in providing opportunities for using it as one entity in a stormwater control plan.

9. The exact future status of the Dundas WPCP is unclear, but it is probable that it will be retained in its present form with the institution of step-feed as an operating policy. The exact impact of this operating strategy upon loadings to Coote's Paradise and pollution control planning for Dundas then only awaits installation of the operating policy and monitoring of its effectiveness.

10. The Waterdown plant is not a major source of flow or loadings to Hamilton Harbour, nor is it impacted by a combined sewer system.

Based upon these points, this Pollution Control Plan study centred upon evaluating the impact of stormwater quality control upon the Woodward Avenue WPCP for options involving storage of combined sewer overflows and diversion of the sewage to the plant after the storm had passed out of the area.

Woodward Avenue WPCP

The Woodward Avenue WPCP is a conventional secondary biological wastewater treatment plant. Sludge is handled by digestion followed by incineration.

The plant is a conventional activated sludge with phosphorus removal attained without chemical addition. Its design capacity is 409 MLD. In 1986, its average day flow was 306 MLD with a quoted (Canviro, 1989) population served of 300,000. More recent data for 1989-90 indicates an average flow of 315 MLD serving a population of 382,000.

The sources of flow to the plant (Canviro, 1989) were attributed to:

- measured industrial sources: 10%
- commercial sources (based upon population x 0.0757: 7 %
- residential sources (based upon population x 0.175): 17 %
- unaccounted for (including infiltration, industrial, commercial and residential sources): 66 %

A profile of industries in catchment is evolving as a part of the MISA monitoring. Based upon the information available in 1987, the following were the main industrial sources.

- Total number of industries 608
- Industries with water 306
- Number of industrial categories 56

The top five types of industry which discharged to the WPCP, (based on water use data) are:

Description	Number
Iron and steel	10
Electrical/Electronic component	17
Metal Finishing	71
Rubber Manufacturing and Processing	5
Coil Coating	2

A process flow schematic of the Woodward Avenue WPCP is shown in Figure 3.4. A summary of the size of each unit is given in Table 3.1.

A summary of the size of each unit is given in Table 3.1. The WPCP is shown in Figure 3.4

Basically, the primary/secondary part of the plant is characterized by two distinct trains of unit operations, generally referred to as the north side and the south side. The north side was the portion of the secondary plant first constructed. The south side represented a major expansion of the plant in 1972 to bring it to its rated capacity of 409 MLD (90 MIGD).

A second unique feature of the plant is that waste activated sludge is returned to the plant influent by connecting into the Redhill Creek interception. It is resettled in the primary clarifiers, from which the "primary sludge" is pumped to the sludge handling train (primary and secondary digesters; and then incineration).

TABLE 3.1

WOODWARD AVENUE WPCP
PHYSICAL SIZES, CAPACITIES AND OPERATIONAL DETAILS

Bar Screens:
- four screens
- each having 2.5 cm openings
- equal influent flow distribution amoung screens assumed

Detritors:
- four detritors
- each detritor: 10.5 m by 10.5 m by 0.76 m
- equal influent flow distribution among detritors assumed

Primary Clarifiers:
- eight primary clarifiers
- each clarifier: 75 m by 12.5 m by 3.3 m
- total clarifier area 7,600 m^2
- equal influent flow distribution among primary clarifiers assumed

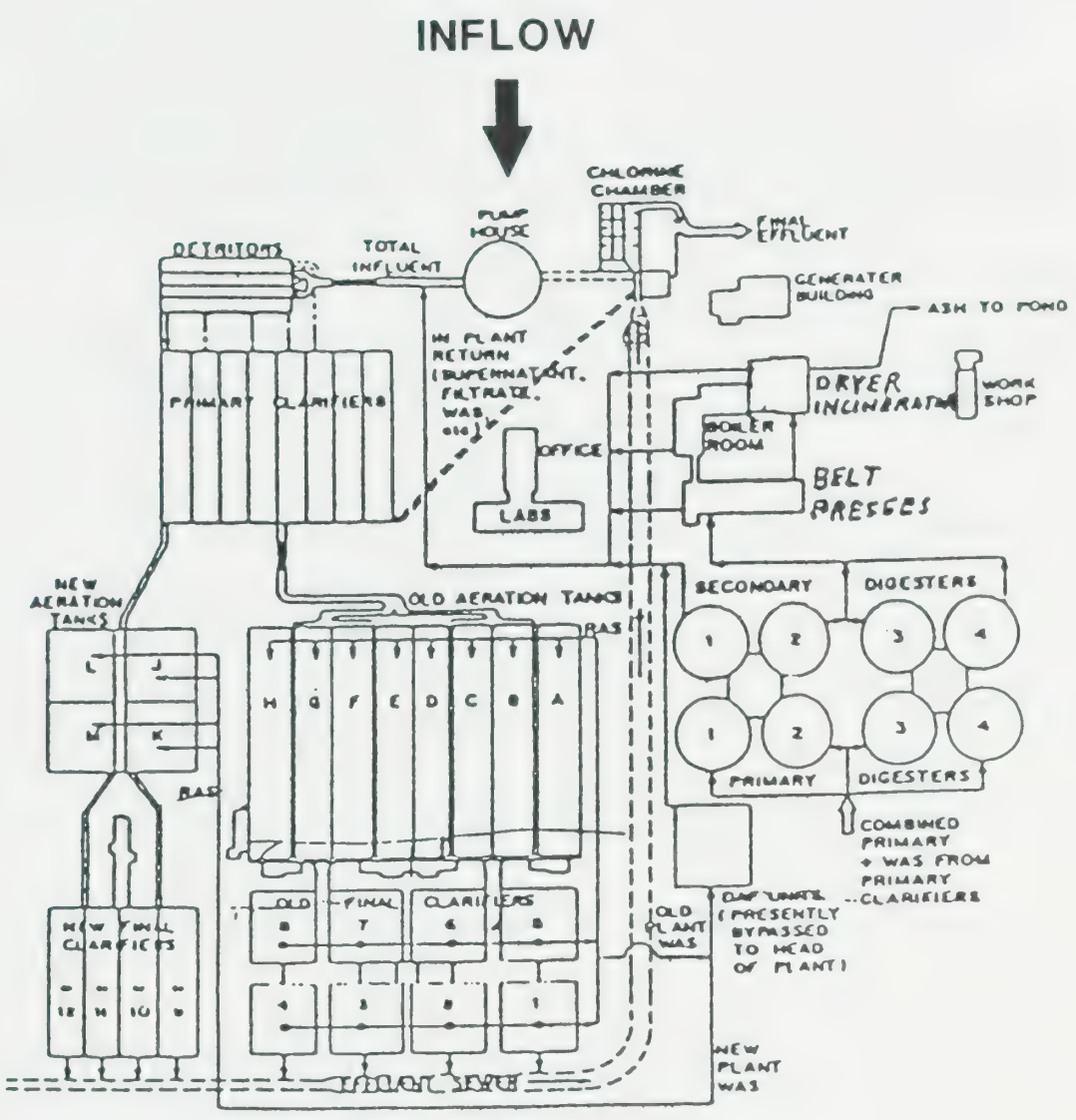
Old (North) Aeration Tanks:
- approximately two-thirds of primary clarified effluent served
- eight aeration rows (two banks of four) having size cells per row; only six rows operated at any one time with only five cells aerated in each
- aeration horsepower: 60 hp per cell
- aeration volume: 18.3 m by 18.3 m by 4.7 m each cell
- equal influent flow distribution among the six rows assumed
- equal return sludge flow distribution among the six rows assumed
- 50 % return rate assumed

Old Final Clarifiers:
- either clarifiers (two banks of four); all in use
- clarification volume: 36.6 m by 36.6 m by 3 m each clarifiers (32,150 m^3 total)
- equal influent flow distribution among clarifiers assumed
- scraper-type sludge collection (each clarifier)
- equal collected sludge flow from each clarifier assumed
- 50 % return rate assumed

New (South) Aeration Tanks:
- approximately one-third clarified effluent servced
- four cells; only two cells in use in fall of 1989
- aeration horsepower: foour @ 60 hp each cell
- aeration volume: 33.5 m by 33.5 m by 4.9 m each cell
- equal influent flow distribution between the two cells assumed
- equal return sludge flow distribution between the two cells assumed
- equal distribution of mixed liquor from both cells to four clarifiers assumed

New Final Clarifiers:
- four clarifiers (two banks of two); all in use in fall, 1989
- clarifiers volume: 73.2 m by 16.6 m by 3.7 m each clarifiers (18,000 m^3 total)
- equal influent flow distribution among clarifiers assumed
- travelling bridge vacuum-type sludge collection (each clarifier)
- equal collected sludge flow from each clarifier assumed
- 50 % return rate assumed

Operational Details:
- MLSS = 2,000 mg/L
- old (north) SRT = 9 days
- new (south) SRT = 6 days



PROCESS FLOW SCHEMATIC OF HAMILTON WPCP

4 – SYSTEM REVIEW

4.1 Introduction

This section presents an overview of the capacities and rainfall response characteristics of the existing combined sewer system. More specifically, this section utilizes historic data to answer the questions listed below.

1. How variable are the flow regulator settings throughout the Region? At which locations are CSO's most/least likely to occur?

2. Does any capacity exist in the sanitary interceptor sewers for the storage and conveyance of additional flows during rainfall events? If additional capacity does exist, then are the flow regulators set to maximize the available storage/conveyance capacity?

3. How do WPCP staff determine the rate at which water is pumped from the wet well to the WPCP? Does sufficient pumping capacity exist in the wet well?

4. During rainfall events, is the WPCP operated to maximize the volume of sewage treated?

5. How is the WPCP operated, and how does it respond, for small/medium and large rainfall events?

6. Can the extent of WPCP bypassing and WPCP inflow be related to rainfall depths?

By addressing these questions a clear understanding of the capacities and response characteristics of the combined sewer system and WPCP can be obtained. This information will be useful when assessing remedial works to control CSO's. Trends and conclusions drawn in this section are based on data from the sources outlined below.

Design Drawings

Numerous plan and profile drawings of the combined trunk sewers, flow regulators and the sanitary interceptor sewers were collected and reviewed. These drawings helped to define the hydraulic characteristics of the sewer system.

Field Investigations

The study team visited each major flow regulator to verify its hydraulic characteristics by taking structural measurements at each overflow structure. This procedure was necessary as the design drawings often contained incomplete details as to flow regulator settings (eg. weir heights).

Flow and Rainfall Monitoring

A flow and rainfall monitoring program was initiated to ascertain the response of the combined sewer system to rainfall events.

WPCP Records

Three years of WPCP records were reviewed to determine the general response characteristics of the WPCP to rainfall events. To complement this data, the study team worked with WPCP staff to initiate a WPCP log book detailing:

- bypass trends at the WPCP;
- the operation of the WPCP controlled sluice gates.

Region Staff

Conversations with numerous technical, process and field staff within the Region helped to clarify the workings of the system.

Rainfall Records

Historic rainfall records were reviewed to ascertain rainfall patterns within the Region.

4.2 System Capacities

Each component of the sewage system has a conveyance capacity that it is limited to. By reviewing these capacities an understanding of the magnitude of flows which are to be expected in the system is obtained, and weak links in the overall system can be identified. Outlined below is a review of

flow/treatment capacities from upstream (i.e. flow regulators) to downstream (i.e. WPCP) sewer system components.

4.2.1 Flow Regulator Capacity

The most upstream components of the sewage system of concern in this study are the flow regulators. Although in excess of 100 regulators exist, only the most significant regulators were considered in the study. The major flow regulators are generally located at the downstream limits of the combined trunk sewers. They consist of weirs, orifice plates, fixed and WPCP controlled sluice gates which regulate combined sewage flows into either the sanitary interceptors or storm outfalls and the transfer of combined sewage between subcatchments. Hydraulic characteristics of the flow regulators were determined using data from detailed design drawings and field surveys.

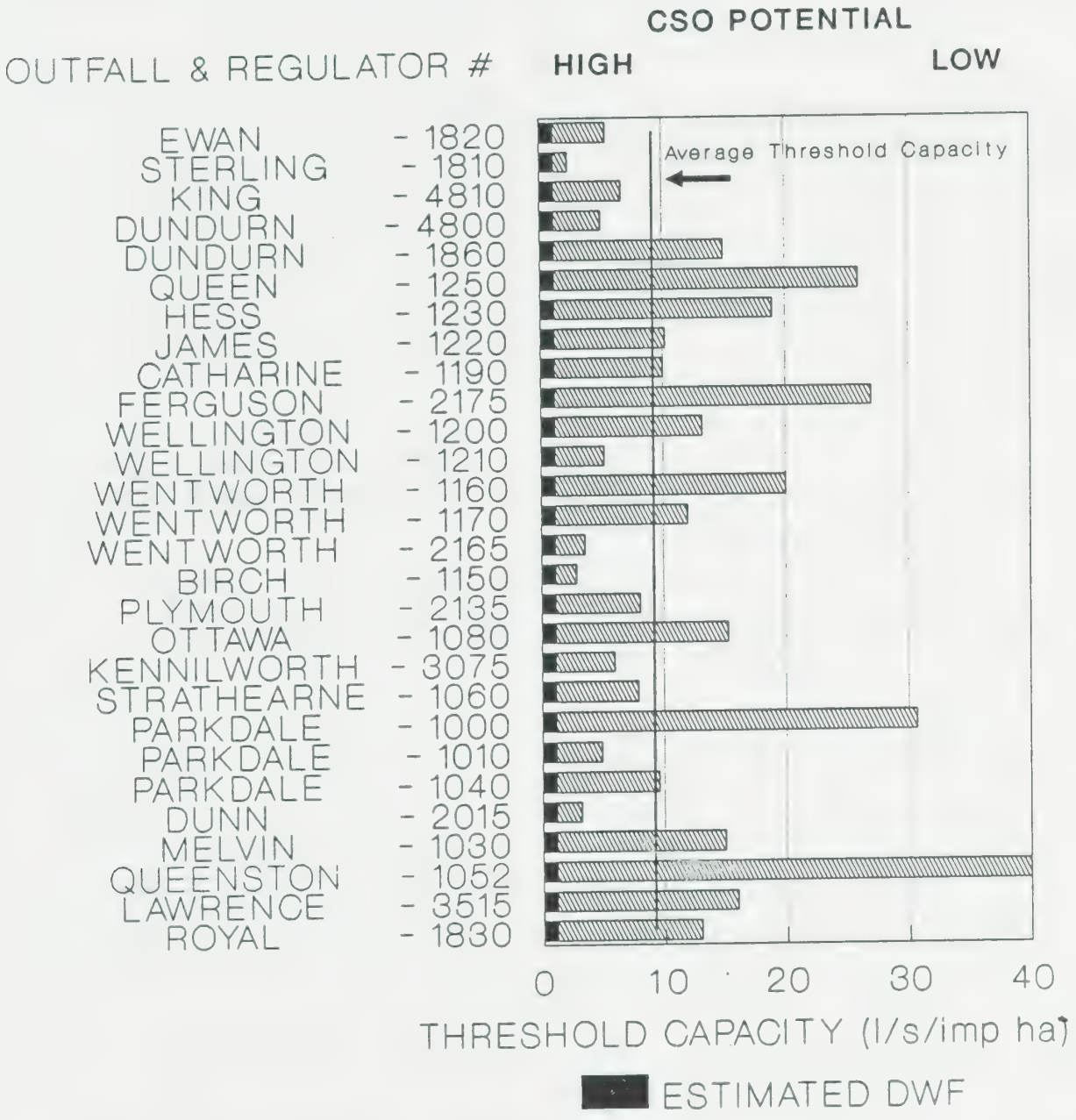
Specific details of the major flow regulators are listed in Tables 1, 2 and 3 in Appendix A. Included in the tables are threshold capacities. Threshold capacities represent the maximum flows on a per (impervious) hectare basis the flow regulators will direct along the dry weather flow drainage path before flow diversions (eg. combined sewer overflows) commence. Impervious, rather that total upstream drainage areas were used because pervious drainage areas (eg. Parks/lawns) rarely contribute stormwater runoff to the combined sewer system during typical rainfall events. Threshold capacities were computed for each flow regulator as they are useful to ascertain where CSO's are most likely to occur.

Figure 4.1 illustrates the threshold capacities of the major flow regulators where CSO's to the receiving water bodies may commence. Included in Figure 4.1 are the anticipated dry weather flows at each of the regulators (expressed as a unit flow per impervious ha of drainage area using observed flows). Overall, a wide range of threshold capacities exist within the Region and dry weather flows generally use approximately 10 percent of the threshold capacity. A review of Figure 4.1 suggests the three most likely outfalls where CSO's may occur are Sterling, Birch and Dunn. Conversely, the three least likely outfalls where CSO's may occur are Ferguson, Parkdale and Queenston.

4.2.2 Sanitary Interceptor Sewer Capacity

Moving downstream, the sanitary interceptor sewers are the next components of the sewer system. By comparing the capacity of these interceptor sewers to upstream flow regulator threshold capacities it is possible to ascertain whether or not the current flow regulator settings (eg. weir heights) over or

FIGURE 4.1: THRESHOLD CAPACITIES OF MAJOR FLOW REGULATORS



Note: The Queenston site has a threshold capacity of 80 l/s/ha.

under utilize the available capacity/storage in the interceptors. Four key locations in the western interceptor were selected for this comparison (Figure 4.2). Locations 1, 2 and 3 are located by Burlington/Gage where sewage flows from 5' x 5'" line (Location 1, servicing waterfront lands west of Gage Avenue) combine with a 90 " line (Location 2, servicing West Hamilton and the downtown core) to be conveyed by a single 90" line (Location 3). Location 4 is located immediately upstream of the WPCP.

As Figure 4.2 illustrates, provided all the WPCP controlled sluice gates are open, more flow is generally expected to flow into the western interceptor than it can convey. This observation suggests the western interceptor's conveyance/storage capacity is well used.

With the WPCP controlled sluice gates closed, however, Figure 4.2 suggests the full conveyance capacity of the western interceptor may not be utilized. This observation is not surprising since the WPCP controlled sluice gates are normally only closed when the WPCP is severely overloaded and requires relief.

A similar analysis was not conducted for the Redhill Creek sanitary interceptor since:

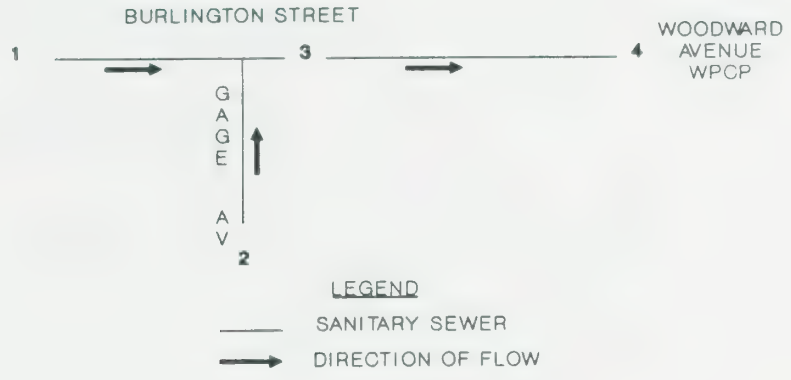
- very few flow regulators contribute to this line; and
- numerous sanitary trunks discharge to this line (eg. the eastern interceptor from Stoney Creek).

Overall, assuming gravity flow, the western and Redhill Creek interceptors can convey a total flow of approximately 1,680 MLD to the Woodward Avenue WPCP. A review of monthly control room reports indicates average and peak dry weather flow rates to the WPCP are 318 and 340 MLD, respectively. Hence, only 20 percent of the capacity and storage of the sanitary interceptors are used during periods of dry weather. This observation suggests additional stormwater runoff may be conveyed and stored by the interceptor sewers.

4.2.3 WPCP Pumphouse Capacity

The pumphouse conveys combined sewage from the sanitary interceptor/wet well to the WPCP treatment processes. WPCP staff determine the pumping rate out of the wet well by taking into consideration the following items.

FIGURE 4.2: COMPARISON OF WESTERN INTERCEPTOR CAPACITY AND FLOW REGULATOR SETTINGS



LOCATION	INTERCEPTOR CAPACITY (cms)	MAXIMUM UPSTREAM INFLOW UNDER CASE 1 (cms)	CASE 2 (cms)
1	2.2	2.4	0.9
2	6.8	5.3	1.1
3	6.8	10.0	4.4
4	9.6	15.4	8.3

CASE 1 - all WPCP controlled sluice gates open
- all regulators operating at threshold capacity

CASE 2 - all WPCP controlled sluice gates closed
- remaining regulators operating at threshold capacity

1. The water level in the wet well.
2. The rate of incoming flows to the wet well.
3. The number of operational pumps in the wet well.
4. Anticipated rainfall volumes.
5. The current operating status of the plant.

An overview of these criteria follows.

The wet well water level and the rate of incoming flows are important factors in determining the pumping rate. Typical water level elevations in the wet well are 63, 67 and 69.5 m for typical low, average and high dry weather flow conditions, respectively. WPCP staff prefer to maintain the water level below elevation 71.5 m as pumphouse flooding commences with a water level of approximately 73 m. Generally, as the wet well water level rises, and the rate of incoming flows increase, the higher the pumping rate out of the wet well.

Six pumps are currently installed in the pumphouse. Each pump has a capacity of approximately 182 to 227 MLD (40 to 50 MGD) yielding a total pumping capacity of 1090 to 1360 MLD (240 to 300 MGD). This pumping capacity is approximately 65 to 80 percent of the maximum inflow to the WPCP of 1,680 MLD (370 MGD). This observation explains how the WPCP can be hydraulically overloaded and why it is sometimes necessary to use the WPCP controlled sluice gates to direct flow away from the WPCP to the receiving water bodies. Conversations with Region staff have also indicated that space exists within the pumphouse to install two additional pumps. A historic review of wet well pumping is included later.

Lastly, anticipated rainfall volumes and the current operating status of the WPCP are used to ascertain an appropriate pumping rate.

4.2.4 WPCP Treatment Capacities

WPCP treatment processes are limited by the maximum hydraulic load the processes can safely handle. Exceeding these limits may result in flooding and/or "washing out" the biological organisms essential to sewage treatment. Regional staff have indicated the following capacities exist at the WPCP.

1. Primary clarifiers (i.e. physical treatment) – 409 MLD
2. Secondary (i.e. biological) treatment – 545 MLD (long term)
   – 615 MLD (short term, 1 to 3 hours).

In addition, Region staff generally operate the WPCP in the manner outlined below during a rainfall event.

1. If possible, the water in the wet well is brought down to provide extra storage.

2. Primary clarifiers are bypassed first under severe inflows. Flows bypassed around the primary clarifiers receive biological treatment.

3. Secondary treatment systems are bypassed before "washout" occurs.

4. The entire WPCP is bypassed and the WPCP controlled sluice gates are closed to direct combined sewage to the receiving water bodies, if severe inflows to the WPCP persist.

This operational procedure suggests the WPCP is operated to maximize the volume of water which is treated by the plant.

A review of the treatment capacities indicates that, unlike most WPCP's, the Woodward Avenue WPCP has a very limited primary treatment process. Generally, primary treatment systems have 1.5 to 2.0 times the capacity of the secondary treatment system, allowing intense inflows to a WPCP to receive at least primary treatment. Furthermore, with average and peak dry weather inflows of 318 and 340 MLD, respectively, and a maximum primary treatment capacity of 409 MLD, little capacity remains to treat additional stormwater inflows.

4.2.5 Summary

The preceding sections have provided useful information on the capacities of the various components of the sewer infrastructure. Overall conclusions that may be drawn from the review are outlined below.

1. With all major flow regulators operating at threshold capacity, sufficient flow is discharged to the western interceptor to utilize its capacity. The existing infrastructure, therefore, attempts to maximize the storage/conveyance capacity of the western interceptor.

2. Closing the WPCP controlled sluice gates to direct combined sewage to the receiving water bodies results in approximately 50 percent less inflow to the western interceptor. This capability helps to decrease the hydraulic load on the WPCP during severe rainfall events.

3. Without the capability to close the WPCP controlled sluice gates, the wet well may flood, since pumphouse capacity is less than interceptor sewer capacity.

4. The pumping capacity of the wet well pumphouse is approximately two to three times the capacity of the WPCP processes. A high pumping capacity is required to prevent pumphouse flooding.

5. Flows pumped from the wet well that exceed WPCP treatment capacities are bypassed.

6. The primary treatment system has very little spare capacity to treat the additional flows which accompany rainfall events.

7. The WPCP is operated to maximize the treatment volume of combined sewage.

4.3 Flow and Rainfall Monitoring

Five flow monitors and two rain gauges were installed to collect flow and rain data for the Pollution Control Plan study. To meet the following objectives, the flow monitors were located at (see Figure 4.3):

1. Barton and Queen (1,500 mm/60 inch sanitary sewer) in order to provide dry and wet weather flow data indicating the magnitude of the flows exiting West Hamilton. Data from this site may indicate if isolated storms in West Hamilton make full use of the capacity available in the western interceptor. The western interceptor is defined as the sanitary interceptor that originates near Highway 403 and travels east along Barton, Ferrie and Burlington Streets to the WPCP.

2. Birch and Brant (2,250 mm/90 inch sanitary sewer) to indicate the magnitude of the flows in the western interceptor within the central business district. The Ottawa Street area was originally chosen for this installation; however, extreme sewer depths and unsafe installation conditions necessitated moving the site upstream to the Brant and Birch area.

REGIONAL MUNICIPALITY OF HAMILTON - WENTWORTH

POLLUTION CONTROL PLAN

LEGEND

- Rainfall Gauging Station
- Flow Monitor
- Sanitary Sewer System
- Storm Sewer System
- Combined Sewer System

FLOW MONITOR AND RAIN GAUGE LOCATIONS

Sites downstream of the Ottawa Street area were not considered because of deeper sewer depths and the possibility of frequent backwater in the western interceptor.

3. Strathearne Avenue (7' x 7-1/2') combined sewer at Vansitmart to ascertain the flows in a representative combined sewershed. The Strathearne/Vansitmart site is located upstream of the WPCP controlled flow regulator at Strathearne and Brampton. It is, therefore, well positioned to determine inflows to the western interceptor, and to estimate the potential for CSO at the Strathearne/Brampton regulator.

4. Fennell Avenue and East 25th Street (8 x 8-1/2' combined sewer) to monitor the response of the (mountain) combined sewers to rainfall. Site 4 was originally selected near Fennell and Upper Ottawa in order to gauge the majority of the mountain. However, excessive flow velocities and deep sewer depths necessitated moving the site upstream.

5. Plymouth Street Storm Outfall (twin 9-1/2' x 7' storm relief sewers) to ascertain the magnitude of CSO to Hamilton Harbour from the central business district during rainfall events. The drainage area upstream of the site has numerous local (i.e automatic) flow regulators, but no WPCP controlled regulators. Flow monitoring data from the outfall will, therefore, indicate the frequency and magnitude of CSO's as a result of local sewer hydraulics. This site was identified by James and Robinson as a major source of pollutants to Hamilton Harbour.

A sixth flow monitor was installed in the Redhill Creek sanitary trunk sewer (near Lawrence Avenue and King Street), but had to be removed shortly after installation because excessive flow velocities would not permit accurate discharge measurements to be taken.

Data was also collected from the WPCP detailing wet well pumphouse operation, WPCP bypassing and the inflow rate to the WPCP processes.

Collectively, flow data from these sites and the WPCP will:

- ascertain typical dry and wet weather flow rates within the western and Fennell interceptors;
- indicate if additional capacity exists in the western interceptor during rainfall events;
- indicate the frequency, and the magnitude, of CSO's at selected outfalls;
- detail WPCP inflows during dry and wet weather;

- provide ballpark estimates of the treatment rates required to handle different sized rainfall events; and
- establish a database of flow data which can be used to calibrate/validate a sewershed model.

The flow monitors installed at sites 1 through 4 (Montedoro Whitney WDFM–8 and Q – Logger models) monitor both depth and velocity. These monitors are, therefore, able to provide accurate discharge measurements during both gravity and surcharged flow conditions. For example, Figure 4.4 illustrates the depth, velocity and discharge profiles at the Birch and Brant monitoring site for the event of July 20, 1989. During this event, backwater effects restricted the flow, allowing the sewer to surcharge for a five hour period. Conventional (i.e. depth only) flow monitors would not be able to monitor the reduction in flow due to backwater effects. Site 5 had a depth velocity gauge from June 8 to July 25 and a depth only gauge from July 26 to August 13.

Tipping bucket rain gauges were installed at:

1. St. Anne's School (near Barton and Sherman).
2. Blessed Sacrament School (near Fennell Avenue and East 37th Street).

These sites (Sites 4 and 6 on Figure 4.3) were selected to collect rainfall data in areas of Hamilton that are currently ungauged. Site 4 (St. Anne's) collected rainfall data in the central business district, while Site 6 (Blessed Sacrament) monitored rainfall patterns on the mountain.

Additional rainfall data is also available from the Region. Sites 1, 2, 3 and 7 have tipping bucket rain gauges operated by M. Robinson, and daily rainfall depths are available from the WPCP.

4.3.1 Dry Weather Flow Characteristics

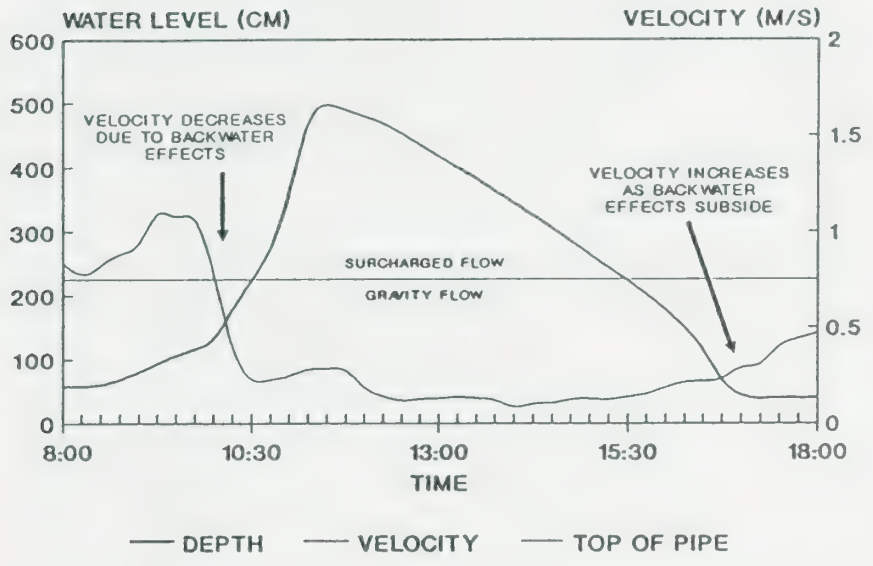
Table 4.1 summarizes the dry weather flows observed during the flow monitoring period. Example flow patterns are illustrated for the western sanitary interceptor (at Birch and Brant) and for the Fennell Avenue combined trunk sewer (at East 25th Street) in Figures 4.5 and 4.6, respectively.

Western Interceptor

Figure 4.5 and Table 4.1 indicate additional flow capacity is available in the western interceptor (at Birch and Brant) during periods of dry weather flow. In fact, only fifteen percent of the sewer's

FIGURE 4.4: OBSERVED FLOW PATTERNS IN WESTERN INTERCEPTOR ON JULY 20, 1989

(LOCATION: BIRCH AND BRANT)



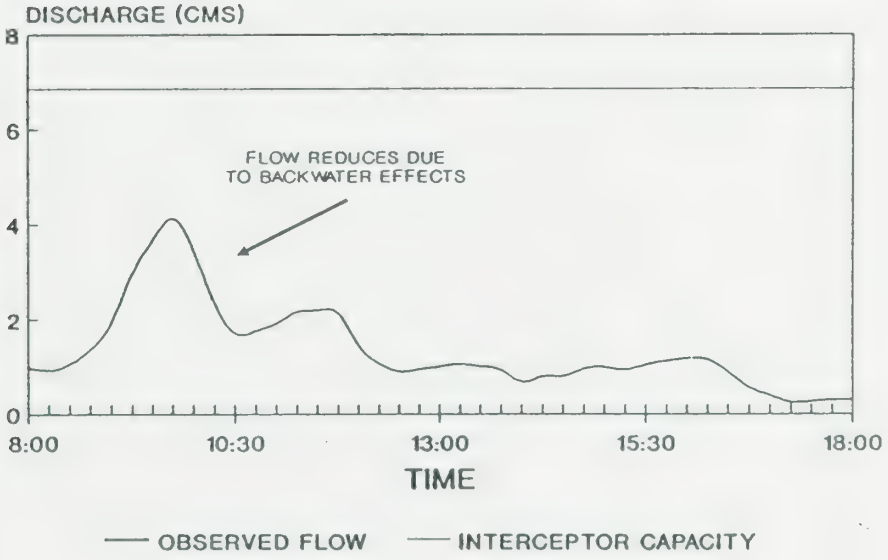
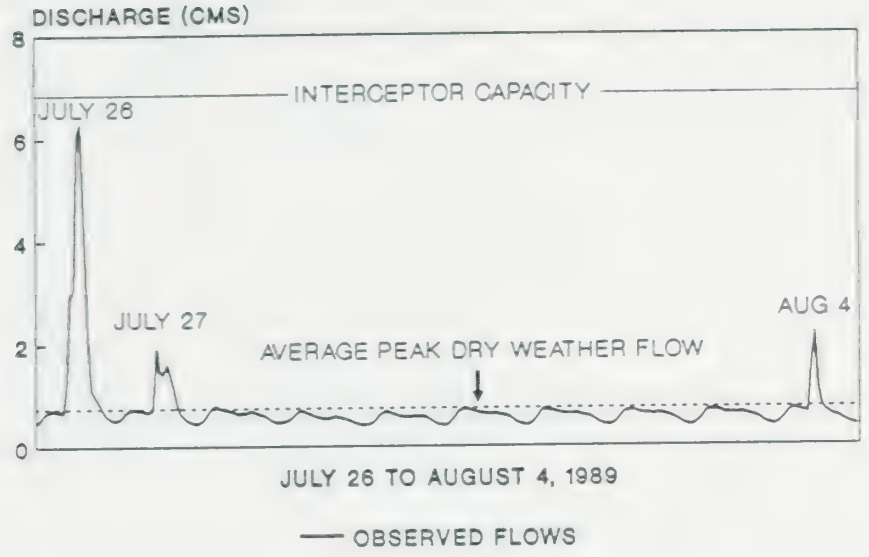


TABLE 4.1

DRY WEATHER FLOW SUMMARY

Location	Sewer		Daily Dry Weather Flows (m^3/sec)		Percent of Sewer Capacity Used By Daily Peak Flow	Flow Depths (cm)		Full Flow Depth (cm)	Percent Of Full Depth Used By Daily Peak Flow
	Type	Capacity (m^3/sec)	Average	Peak		Average	1-Hour Peak		
Queen and Barton	1500 mm sanitary trunk	2.3	0.60	0.8	35	60	70	150	47
Birch and Brant	2250 mm sanitary trunk	6.8	0.70	1.0	15	50	60	225	27
Strathearne and Vansitmart	2.13 m X 2.29 m combined	10.7	0.27	0.36	3.5	11	14	229	6
Fennell and East 25th	2.44 m X 2.59 m combined	16.5	0.50	0.70	4.2	20	25	259	8

FIGURE 4.5: FLOW PATTERNS IN WESTERN INTERCEPTOR AT BIRCH AND BRANT



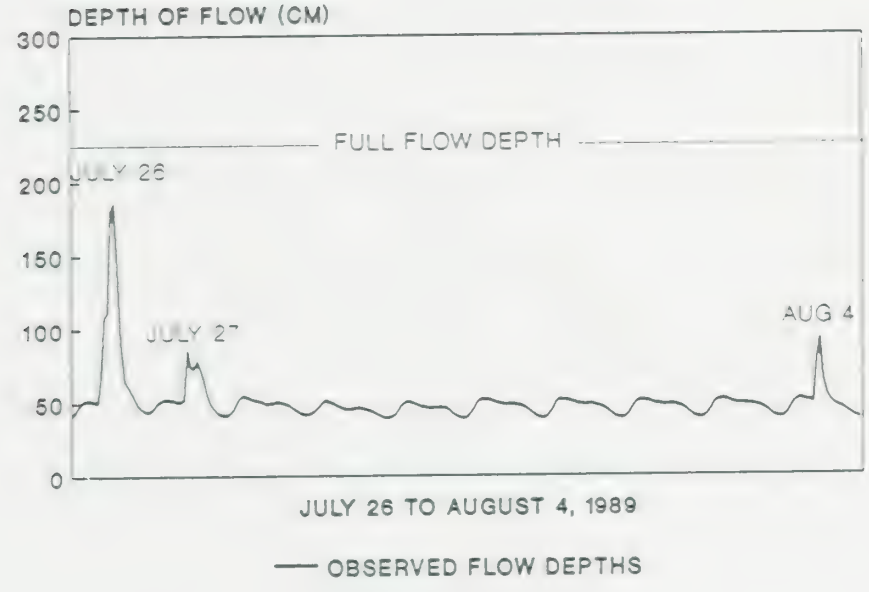
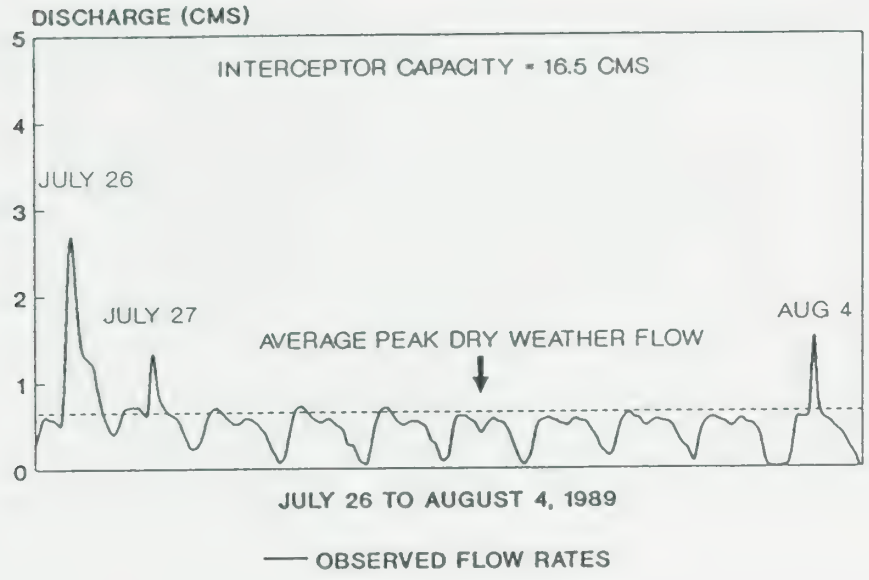
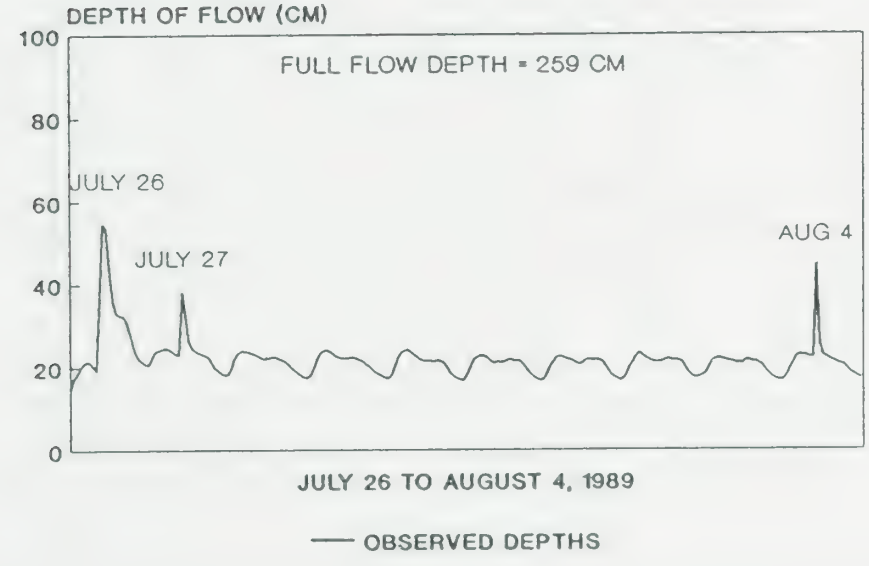


FIGURE 4.6: FLOW PATTERNS IN FENNELL AVE COMBINED TRUNK SEWER AT EAST 25TH





capacity is used by the average daily peak flow. The capacity of the western interceptor to accommodate stormwater flows is illustrated in Figure 4.5 for the rainfall events of July 26, July 27 and August 4, 1989. These events, which averaged 7.2 mm in depth, did not require the full capacity of the western interceptor (at Birch and Brant) to convey the combined sewage flows. These events are discussed in more detail in Section 4.3.2.

Fennell Avenue Interceptor

Dry weather flows in the Fennell Avenue interceptor (at East 25th Street) require less than five percent of the sewer's capacity. Figure 4.6 illustrates the typical dry weather flow pattern (zero flows in Figure 4.6 are the result of inadequate flow depths and/or temporary sewage buildup which prevented accurate velocity measurements in the evenings). The additional capacity available for stormwater flows is greater for the Fennell combined interceptor than the western interceptor because combined sewers are designed to carry all of the stormwater runoff and sewage. Conversely, sanitary interceptors are designed to carry only a fraction of the total combined flow during a rainfall event. Figure 4.6 also indicates that the combined flows on the Fennell Avenue interceptor during the events of July 26, 27 and August 4 were not severe.

Strathearne Avenue Combined Trunk Sewer

Dry weather flows in the Strathearne Avenue combined trunk sewer are very low relative to the sewer's capacity. The drainage area upstream of the flow monitoring site covers approximately 75 percent of the area that drains to the WPCP controlled sluice gate regulator at Strathearne/Brampton. The average dry weather flow from this area requires less than 6 percent of the capacity of the sanitary sewer that conveys sewage from the Strathearne/Brampton regulator to the western interceptor. This observation suggests that substantial stormwater inflow to the western interceptor can occur at this regulator.

4.3.2 Wet Weather Flow Characteristics

Classification of Events

In order to compare the effects of rainfall events on both the sewer systems and the WPCP, a means of classifying the rainfall events was required. Rainfall events were classified by:

1. Determining the severity of a rainfall event using rainfall depth, duration and intensities. Past rainfall trends were also used to classify the events. Figure 4.7 summarizes the daily rainfall depths at the WPCP from January 1986 to May 1989. A review of Figure 4.7 suggests that rainfall events may be classified, generally, as either small (1 - 4 mm), medium (4.1 - 12 mm) or large (greater than 12.1 mm).

2. Noting the treatment rates at the WPCP. The average 12-hour treatment rate (commencing from the start of the storm) and the peak 1-hour treatment rate were used. Treatment rates were extracted from daily WPCP control room reports.

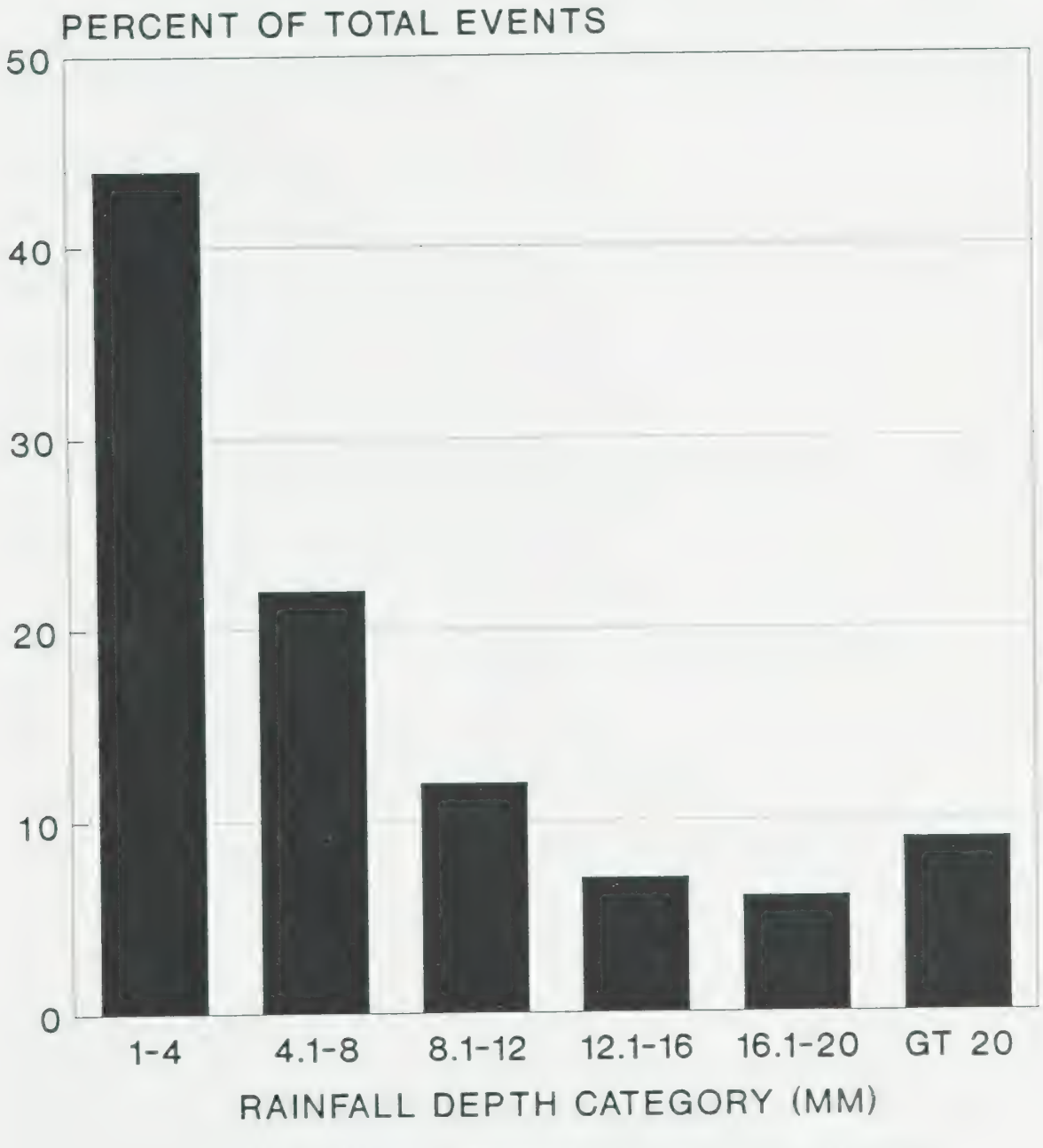
3. Categorizing the response of the Woodward Avenue WPCP to rainfall events. The categories adopted were:

   i) No bypasses (events did not overload the WPCP): As a result, no intentional sewage diversions were initiated by WPCP personnel. These events generally cause a loading on the WPCP less than or equal to its treatment capacity of approximately 90 mgd.

   ii) Internal bypass events: WPCP inflows during these events exceeded the treatment capacity of the WPCP. As a result, WPCP personnel are forced to initiate primary and/or secondary bypasses within the WPCP. Primary bypasses route sewage around the primary clarifiers to the aeration tanks. Secondary bypasses route sewage away from the aeration tanks, through a chlorination chamber, and to Hamilton Harbour.

   iii) Plant/sewer bypass events represent the most severe loads on the Woodward Avenue WPCP. Sewer bypasses (i.e. the closing of WPCP controlled sluice gates at Royal Avenue, Strathearne, etc.) are usually initiated by the WPCP to lessen the hydraulic load on the plant if internal bypasses have not provided enough relief. Combined sewage discharged to the receiving water bodies from sewer bypasses are untreated. Plant bypasses are usually initiated as a last resort to lower the load on the WPCP. Plant bypasses route sewage from the wet well to a chlorination tank and to Hamilton Harbour. Plant/sewer bypasses are lumped together into one category because WPCP monthly summary sheets lump the hours of plant/sewer bypasses together. At our request, the WPCP commenced a more detailed logging of plant/sewer bypasses in July. Internal bypasses usually occur in conjunction with plant/sewer bypasses.

FIGURE 4.7: BREAKDOWN OF DAILY RAINFALL DEPTHS FROM 1986-1989



(RAIN GAUGE: WOODWARD AVENUE WPCP)

These bypass categories were constructed after a preliminary review of WPCP records and conversations with WPCP personnel.

4. Noting whether or not CSO's occurred at the Plymouth Street outfall. Estimates of CSO quantities could not be made at this site due to questionable flow measurements caused by harbour backwater and the opening/closing of a tide gate upstream of the site.

5. Assessing the magnitude of the flows produced by the rainfall within the sewer systems.

Observed Wet Weather Flows

Wet weather flow characteristics for the observed rainfall events are summarized in Table 4.2. A discussion of the trends of small, medium and large rainfall events follows.

Small Rainfall Events

No bypasses were initiated by the WPCP for small rainfall events. The small rainfall events are characterized by rainfall depths not exceeding 4.0 mm and very mild rainfall intensities. The general trends observed during these events were:

1. The 12-hour average WPCP treatment rate was less than the WPCP treatment capacity (hence no bypasses).

2. CSO's occurred at the Plymouth Street Outfall during 50 percent of the events.

3. The western interceptor at Birch and Brant was, on the average, only 40 percent full.

4. Small flows were observed in the Fennell interceptor.

These observations suggest that local overflows (i.e. weirs/sluice gates) are directing combined sewage to the storm relief sewers when the capacity exists in the western interceptor to convey additional flows to the WPCP for treatment. CSO from the small events could, therefore, be reduced, or perhaps eliminated, by redirecting more combined sewage to the western interceptor. This could be accomplished by adjusting the overflow regulators. Considering that approximately 45 percent of daily rainfall depths do not exceed 4 mm in depth (see Figure 4.7), a substantial reduction in the

TABLE 4.2

WET WEATHER FLOW CHARACTERISTICS

Date	Average Rainfall			Peak 1-hr. Rainfall Intensity (mm/hr)	WPCP Treatment Rates (MLD)		Duration of WPCP Bypassing		Did CSO's occur at the Plymouth Street Outfall	Flow Conditions in Interceptors[4]	
	Depth (mm)	Duration (hr)	Intensity (mm/hr)		12-hr. Average[1]	1-hr. Peak	Internal[2]	Plant and Sewer[3]		Fennel (@ East 25th)	Western (@ Birch & Brant)
Small											
July 12	1.4	2.6	0.5	1.0	364	387	0	0	0	0.08	0.3
Sept. 8	1.7	0.17	10.0	1.7	305	319	0	0	N.A.	0.15	0.5
June 21	2.2	4.6	0.5	1.2	391	410	0	0	N.A.	N.A.	N.A.
July 25	2.8	2.0	1.4	1.8	350	400	0	0	No	0.17	0.4
June 17	3.3	1.1	2.8	3.2	337	387	0	0	No	N.A.	0.4
July 4	3.8	2.4	1.6	2.1	410	432	0	0	Yes	0.14	0.4
June 16	4.0	6.2	0.7	2.3	323	387	0	0	Yes	N.A.	0.4
Medium											
July 27	5.0	1.3	4.0	4.5	419	437	1.5	0	N.A.	0.21	0.4
June 24	5.7	1.6	3.5	4.0	378	391	3	0	Yes	0.12	0.4
Aug. 4	5.7	1.0	5.7	5.7	373	410	3.0	0	Yes	0.22	0.5
Sept.16	6.8	10.0	0.7	1.3	400	500	N.A.	N.A.	N.A.	0.14	N.A.
June 19/20	7.7	10.9	0.7	3.9	467	546	21	0	Yes	N.A.	0.5
July 26	11.0	3.5	3.1	10.4	515	614	7.1	0	N.A.	0.24	0.9
Large											
Aug. 15/16	14.6	6.1	2.4	10.0	482	614	8.5	0	Yes	0.39	surch'ed
Sept.14	18.9	7.2	2.6	4.0	469	614	N.A.	N.A.	N.A.	0.24	N.A.
June 22	20.0	1.0	20.0	20.0	528	637	15	0	Yes	N.A.	1.0
July 20	21.6	8.0	2.7	8.1	555	660	11	8.5	Yes	0.27	surch'ed
Sept. 1	31.6	10.1	3.1	14.3	482	628	15.0	3.7	N.A.	0.44	surch'ed

1 From start of event

2 Primary and/or secondary by-passes within the Woodward WPCP controlled regulators

3 Plant by-passes and/or diversion of sewage at the STP controlled regulators

4 (Peak flow depth)/(Full flow depth)

frequency of CSO may be possible with only minor modifications of the existing sewer system. This possibility is addressed as a remedial option in Section 9.

Medium Rainfall Events

Medium sized rainfall events (i.e. the 5 to 11 mm events) averaged 7 mm in depth, had more intense rainfall intensities than the small events, and had the following effects on the sewer systems and the WPCP:

1. Internal WPCP bypasses were required to lessen the hydraulic load on the WPCP.

2. CSO's occurred at the Plymouth Street outfall.

3. Larger volumes of water entered the western interceptor, filling it to an average depth of 54 percent full.

4. Inflows to the Fennell interceptor exceeded those observed for the small events, although substantial flow capacity was still available in the interceptor.

Combined, the small and medium sized rainfall events accounted for approximately 75 percent of the monitored events. For these events, the WPCP did not have to intentionally divert sewage away from the Woodward Avenue WPCP using the nine WPCP controlled regulators. Therefore, the sewer system directed the maximum possible inflow to the WPCP and this, generally, resulted in little or no internal bypassing around the primary clarifiers or the aeration tanks. This trend is consistent with the views of WPCP personnel, who indicated that activation of the WPCP controlled regulators and plant bypasses are only required during extreme events to prevent hydraulic overloading of the WPCP.

Large Rainfall Events

Five large rainfall events were also observed during the flow monitoring period. These events ranged from 15 to 30 mm in depth and, overall, had the most extreme rainfall intensities. The following trends were noted during these events.

1. Prolonged internal bypassing was required to protect the WPCP from hydraulic overload.

Sewer bypasses at the WPCP controlled regulators were also required for the two most extreme events. In addition to sewer bypasses, plant bypassing was required for the September 1 event.

2. Large CSO volumes appeared to pass through the Plymouth Street outfall.

3. The full capacity/storage of the western interceptor was used at the Birch and Brant site.

4. The Fennell Avenue interceptor was less than half full at East 25th Street.

The heavy hydraulic loads placed on the WPCP during these events indicate the existing sewer system can convey large volumes of combined sewage to the WPCP. Unfortunately, the WPCP capacity is limited and extensive bypassing is required to protect the plant from hydraulic overload. Primary, secondary and plant bypasses are chlorinated and, hence, receive some treatment. Combined sewage directed to the receiving water bodies by the WPCP controlled regulators receive no treatment whatsoever.

The preceding analysis indicates the flow and rainfall monitoring program has helped to assess the severity of CSO's within the Region. Bypass trends will be analyzed further in Section 4, using long-term WPCP records.

4.4 Review of Woodward Avenue WPCP Records

The response of the Woodward Avenue WPCP to rainfall events was ascertained through an analysis of January 1986 to June 1989 monthly summary sheets and daily 1989 WPCP process and wet well pumphouse records. The data was analyzed to determine:

- wet well pumping;
- WPCP inflows; and
- bypass trends

at the WPCP during rainfall events. This data may subsequently be used to determine approximate treatment rates required at the WPCP for different sized rainfall events. An overview of these items follows.

4.4.1 Wet Well Pumping

Six pumps are currently installed in the wet well pumphouse. Table 4.3 summarizes the results of a review of wet well pumphouse records. This review indicates:

1. Two pumps are required to handle dry weather inflows to the WPCP.
2. Three or four pumps are sufficient to handle inflows from the majority of rainfall events.
3. Five pumps are required to handle WPCP inflows during extreme rainfall events.

These observations suggest, under current conditions, minimum and maximum WPCP treatment capacities of 545 MLD (3 pumps x 182 MLD/pump) and 910 MLD (4 pumps x 227 MLD/pump) are required to provide full treatment of WPCP inflows for most rainfall events.

TABLE 4.3

TYPICAL PUMP OPERATION AT THE WET WELL PUMPHOUSE

Date (1989)	Rainfall Depth (mm)	Maximum # of Pumps Operating	Maximum Water Level In Wet Well (m)
Dry Weather Flow	–	2	69.5
Aug. 15/16	14.6	3	69.8
Sept. 14	18.9	4	70.7
June 22	20.0	3	70.4
July 20	21.6	3	72.8
Sept. 1	31.6	4	71.0
Sept. 22/23 (Hugo)	40.0+	5	71.6

Note: Wet well flooding commences at Wet Well Water Level of approximately 73 m.

4.4.2 Inflows During Rainfall Events

The influence of rainfall on WPCP inflows was assessed further by ascertaining the increase in WPCP inflows during rainfall events. This assessment included determining the average increase in WPCP inflow, above DWF conditions, that various rainfall events induced on the WPCP. The procedure used to compute the increase in inflow is outlined below.

12-Hr. Average WPCP Treatment Rate	-	12-Hr. Average WPCP Treatment Rate Under Dry Weather Flow Conditions	=	12-Hr Average Increase in WPCP Inflow During Rainfall Event

The 12 hours following the start of rainfall was chosen as the period of analysis based on a preliminary screening of increases WPCP inflows due to rainfall events.

Figure 4.8 illustrates the results. Overall, a well defined trend exists. WPCP inflows increase, on the average, 11 MLD (hourly rate) per mm of rainfall for the 12 hours following a rainfall event. The results of this analysis provide some insight as to the additional treatment capacity required at the WPCP in order to treat the combined sewage from various rainfall events. For example, increasing the primary treatment capacity to 550 MLD would allow the current system to treat WPCP inflows for all events up to approximately 20 mm in depth.

4.4.3 Bypass Trends

The review of monthly WPCP summary sheets indicated rainfall events which:

- resulted in no bypasses;
- resulted in primary and/or secondary bypasses (internal WPCP bypasses); and
- resulted in internal bypasses with plant bypasses and/or gate closures (i.e. plant/sewer bypasses).

Events were further subdivided using rainfall depths. Rainfall depth intervals of 1-4, 4.1 -8, 8.1 - 12, 12.1 - 16, 16.1 - 20 mm and greater than 20.1 mm were used to categorize the severity of the rainfall events. A total of 304 events were reviewed.

Events which were not used in the review included bypass events with no rainfall record, events with less than 1 mm of rainfall and bypass events due to excessive snowmelt.

Figure 4.9, which breaks down the percentage of rainfall events in each depth interval that required either no, internal, or plant/sewer bypasses, illustrates the observed bypass trends. For small (i.e. 1 to 4 mm) rainfall events, sewage bypasses are not normally required. However, as the "No Bypass" chart in Figure 4.9 indicates, once rainfall events exceed 4 mm, the probability of operating the WPCP without bypassing is small. The "Internal ByPass" chart indicates internal bypassing is usually

FIGURE 4.8: OBSERVED INCREASE IN WOODWARD AVE WPCP INFLOWS DURING RAINFALL EVENTS

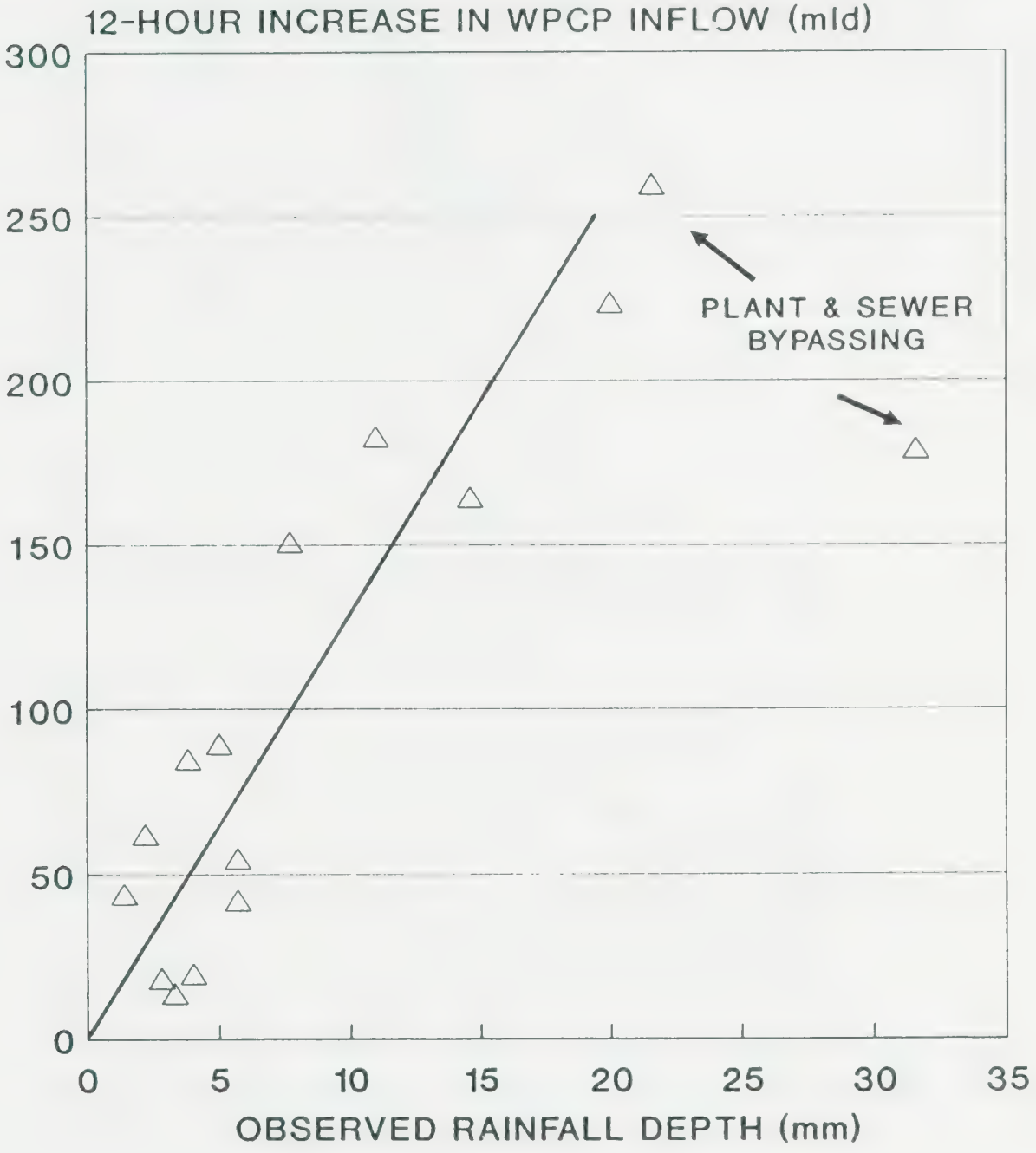
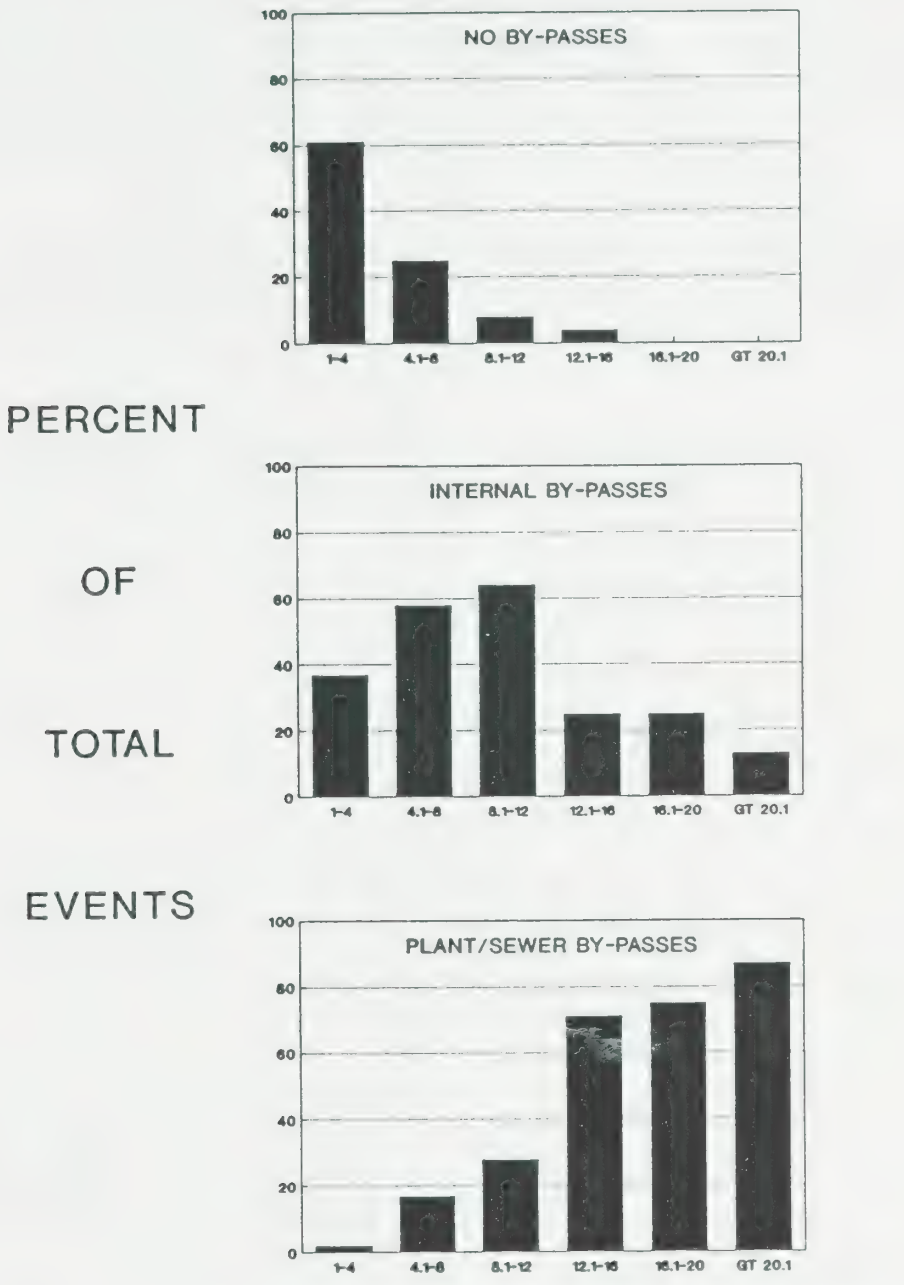


FIGURE 4.9: OBSERVED BY-PASS TRENDS AT THE WOODWARD AVENUE WPCP (1986-1989)



required to handle WPCP inflows during medium (i.e. 4.1 to 12 mm) sized events. Events exceeding 12 mm in depth have required plant/sewer bypasses at least 70 percent of the time.

Table 4.4 provides a summary of 304 bypass events reviewed in this analysis. No bypass events averaged 2.8 mm in depth, and accounted for approximately one third of all events. Internal bypass events accounted for 41 percent of the events, and averaged 6.9 mm in depth. Plant/sewer bypass events averaged 17.7 mm in depth, and represented 24 percent of all events.

The preceding analysis illustrates the trends of sewage bypass events. It can not, however, provide definite limits as to when sewage bypasses may or may not be required to protect the WPCP from hydraulic overload because:

1. The observed rainfall depth at the WPCP is not necessarily equal to the average rainfall depth throughout the WPCP's sewershed.

2. Other factors such as storm duration and peak rainfall intensity will affect inflows to the WPCP.

To complement Figure 4.9, the bypass events observed during the flow monitoring period, i.e. Table 4.2, are illustrated in Figure 4.10. Rainfall depths for these events are based on rainfall data from gauging stations 1, 2, 3, 4, 5, 6 and 7 (see Figure 4.3) and are more representative of the average rainfall depths throughout the WPCP's sewershed. Most noticeable from Figure 4.10 is that events which exceeded 4 mm required bypassing. Internal bypass events ranged from 5 to approximately 20 mm. The two events which required plant/sewer bypasses exceeded 20 mm in depth.

Overall, the long term bypass trends using only WPCP rainfall data, and the short term observations using data from seven rain gauges show similar trends. Placing more weight in the long-term observations in Figure 4.9 yields the trends outlined below.

1. Bypasses are not required for approximately one third of all rainfall events and these events ranged from 1 to 4 mm in depth.

2. Internal bypassing at the WPCP is usually required for events ranging from 4.1 to 12 mm.

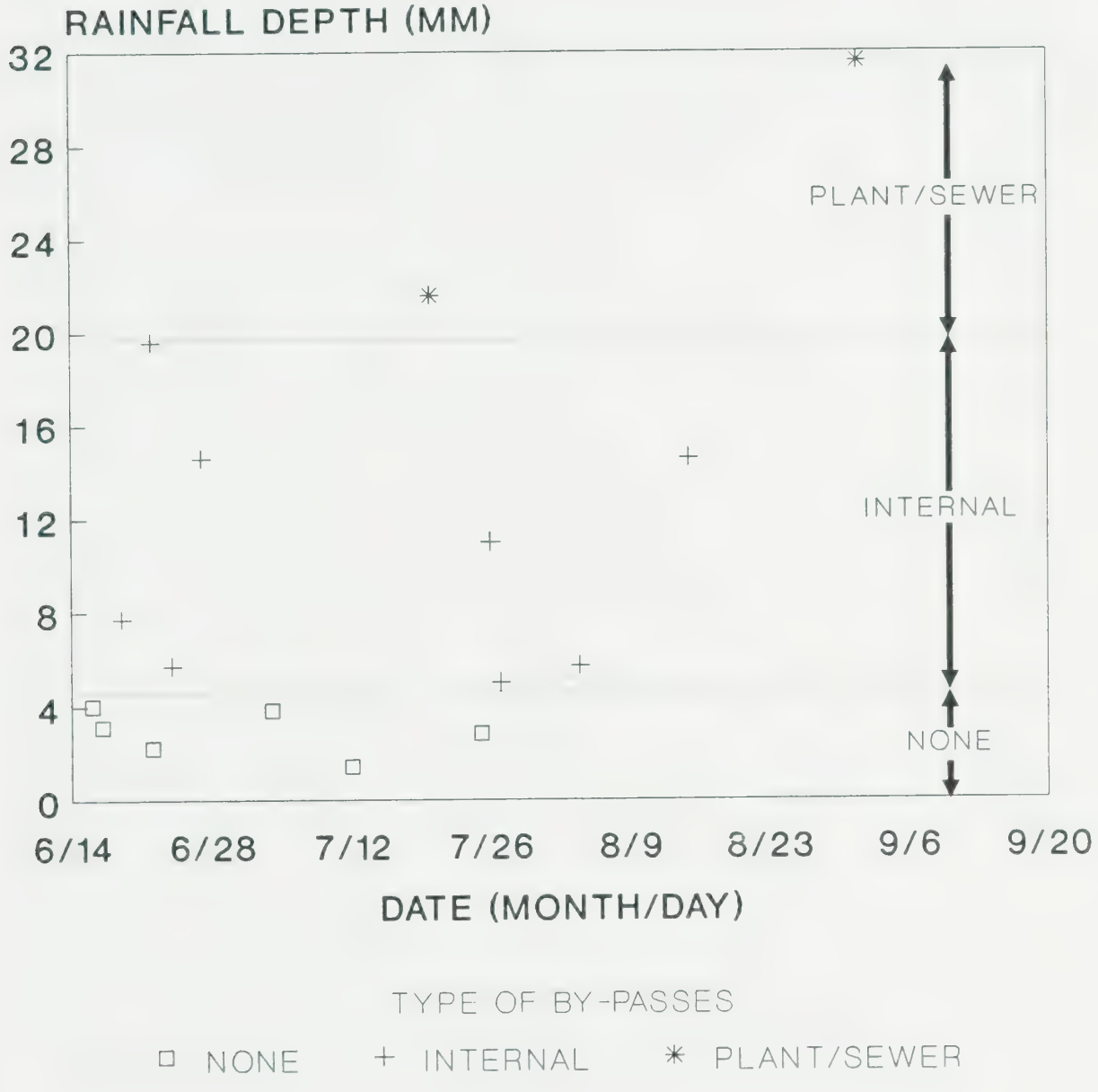
TABLE 4.4

GENERAL CHARACTERISTICS OF

BY-PASS EVENTS

Type of By-Pass	Number of Events Observed*	Percent of By-Pass Events	Average Rainfall Depth** (mm)	Percent of Rainfall Events From January 1986 to May 1989 With Depths Less Than The Average Rainfall Depth
No By-Pass	105	35	2.8	35
Internal By-Pass	125	41	6.9	61
Plant By-Pass and/or Sewer Diversions	74	24	17.7	87
TOTAL	304	100		

FIGURE 4.10: WOODWARD AVE WPCP BY-PASS EVENTS (JUNE TO SEPTEMBER, 1989)



3. Plant/sewer bypasses are not normally required until rainfall depths exceed 12 mm. Furthermore, the rainfall statistics in Figure 4.7 and Table 4.2 indicate that rainfall events less than 8 mm account for approximately two thirds of all rainfall events; reinforcing that plant/sewer bypasses are not normally required to limit WPCP inflows.

4.4.4 Response to Rainfall Events

Further insight as to the response of the WPCP to rainfall events may be obtained by graphically illustrating the response of the WPCP to specific rainfall events. Figure 4.11 illustrates the response of the WPCP to three rainfall events. Each event includes information on the:

i) observed rainfall distribution;
ii) observed inflow gauged at the WPCP; and
iii) normal (i.e. dry weather) WPCP inflow.

An overview of each of these events follows.

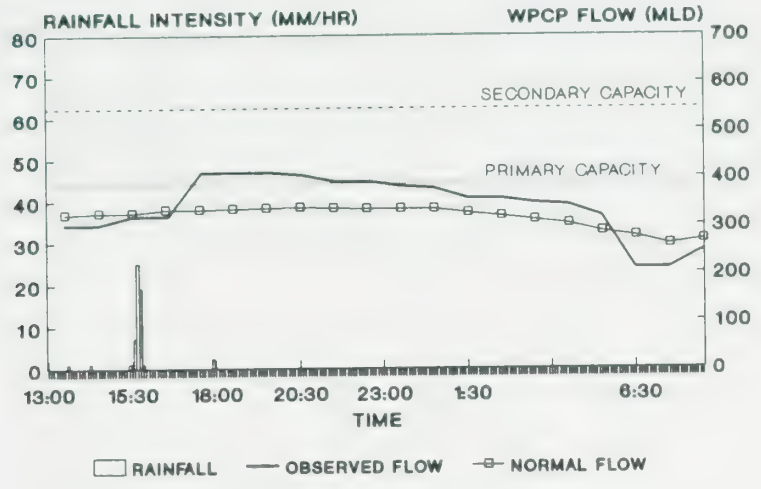
August 4, 1989 (5.7 mm): This short duration storm caused flows to increase rapidly at the WPCP. Approximately 3 hours of internal bypassing around the primary clarifiers was required to protect the units from hydraulic overload. WPCP flows returned to normal after approximately 13 hours.

July 26, 1989 (11.0 mm): This low intensity 3-hour storm required approximately seven hours of primary clarifier bypassing. No bypassing of the secondary treatment system was required due to the short duration of increased WPCP inflows above secondary treatment system capacity. WPCP inflows returned to normal after approximately 11 hours.

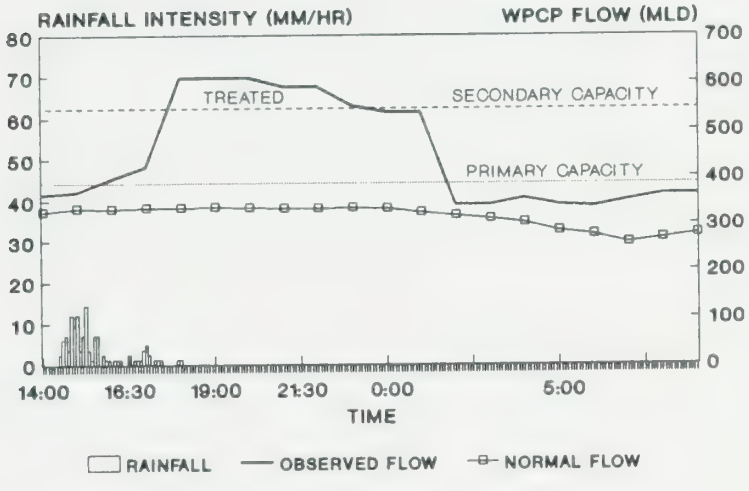
September 1, 1989 (31.6 mm): This event included both large rainfall volumes and high rainfall intensity. The rapid increase in WPCP inflows necessitated 15.0 hours of internal bypassing and, ultimately, approximately 3.5 hours of plant bypassing (note the decrease in WPCP inflows during plant bypassing). A review of WPCP records suggests that approximately 57,000 m^3 of combined sewage was bypassed around the plant. WPCP inflows returned to normal after approximately 14 hours.

A conclusion which may be made based on the preceding analysis is that rainfall events generally increase WPCP inflows for approximately 12 hours.

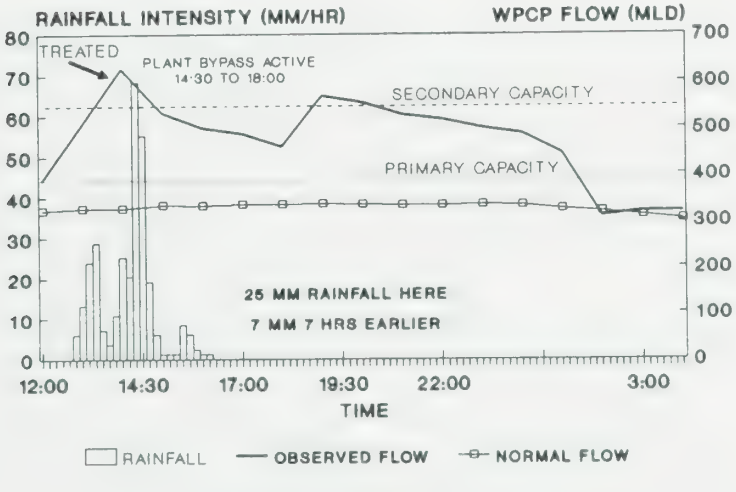
FIGURE 4.11: RESPONSE OF WOODWARD AVENUE WPCP TO RAINFALL EVENTS



a) AUGUST 4, 1989 - 5.7 MM



b) JULY 28, 1989 - 11.0 MM



c) SEPTEMBER 1, 1989 - 31.6 MM

4.4.5 Summary of Hydraulic and Operating Constraints for Woodward WPCP

A summary of the hydraulic capabilities of different elements of the Woodward Avenue WPCP are summarized in Table 4.5, together with the design basis, and current plant operating policies.

TABLE 4.5

SUMMARY OF HYDRAULIC LIMITING FACTORS,

EXISTING PLANT LOADS

1.	Interceptor Sewer		
	Burlington Street Interceptor (102" @ 0.1% = 9.6 cms) =	184 MIGD	830 MLD
	Redhill Creek Interceptor (102" @ 0.13% = 10.0 cms) =	192 MIGD	860 MLD
		376 MIGD	1690 MLD
2.	High Lift Pumps		
	Theoretical 6 @ 50 MIDG = 300 MIGD = 1135 MLD		
	Theoretical 5 @ 50 MIDG = 250 MIGD = 1120 MLD		
	80% of Theory for 5 pumps = 200 MIGD = 900 MLD		
	Assumed Rating for 5 pumps, 1 out of service, and 80% to 100% of rated capacity		900–1120 MLD
3.	Current Plant Hydraulic Loadings		
	• Average Dry Weather Flow (Population 382,000 excluding Dundas) =	70 MIGD	315 MLD
	• Diurnal Curve Operating Range, using interceptor storage as a buffer		
	– Low Flow =	45 MIGD	203 MLD
	– Peak Flow =	78 MIGD	350 MLD
4.	Current Plant Design Basis		
	• Units Designed at – Primary Clarifiers	60 MIGD	270 MLD
	• Units Designed at – Aeration Tanks – Secondary Clarifiers	90 MIGD	405 MLD
	• Design Capacity for Various Connecting Channels – High Lift Pumps to GRT Chambers		150 MIGD
5.	Current Plant Operating Policy with Respect to By–Passing		
	• Primary Clarifier – Bypassed at 80–85 MIGD		360–380 MLD
	• Secondary Treatment Units – north side bypassed when sludge washout starts – south side bypassed when sludge washout starts		
	• Whole plant (flow diverted from High Lift Pump Station) – Bypassed at		
	– short term = 130 MIGD =600 MLD		
	– long term = 120 MIGD =540 MLD		

5 – HYDROLOGIC, HYDRAULIC AND WPCP PROCESS MODELLING

5.1 Introduction

This section provides an overview as to why modelling was required in the study, the selection and development of the models, a review of rainfall and water quality data which was used during modelling, and calibration/validation of the selected models.

5.1.1 Required Level of Analysis/Modelling

At the onset of any pollution control planning study the decision has to be made as to the level of analysis/modelling required to meet the study objectives. In this study consideration must be given to each of the major components of the sewage system – combined sewer areas, separated/natural drainage areas, and the WPCP. Generally, five different model types are available for analysis, namely:

1. Desk–top calculations;
2. Spreadsheet Models;
3. Event models;
4. Continuous models; and
5. Planning models.

Each of these model types were used during the study. The selection of the model(s) for each component of the sewage system was based on general study objectives, and is outlined below.

5.2 Modelling Objectives and Model Selection

5.2.1 Combined Sewer System

A review of the Terms of Reference suggests the selection of a model for combined sewer system analysis may be based on three general objectives:

1. Ascertain annual CSO volumes and the resulting constituent loads to the receiving water bodies.

2. Assess, if possible, any proposed remedial works to control CSO's.

3. Determine the response of the combined sewer system under both continuous and event situations.

Consideration of these objectives indicate a comprehensive continuous/event based urban drainage model is required for proper analysis of the system for the reasons outlined below.

1. Annual constituent loadings to the receiving water bodies from CSO's will be influenced by the runoff volumes generated from combined sewersheds. Although runoff characteristics are fairly constant for impervious surfaces within the sewersheds, runoff volumes for the pervious surfaces vary. The factors which most influence runoff characteristics on pervious surfaces – interevent time and soil moisture – may be accounted for by continuous modelling.

2. The effectiveness of any proposed facility to capture and store CSO's for subsequent treatment will be dependent on interevent time and the drainage rate of the facility. These factors will influence the volume and frequency of CSO's from a facility. Each factor can be incorporated in a continuous model in order to achieve an optimum design.

For these reasons the United States Environmental Protection Agency's Stormwater Management Model (SWMM) was selected to meet the study objectives. SWMM is a powerful and flexible urban drainage system model which can be used in either an event or a continuous manner. Specific details on the features of SWMM are included in Section 5.3.1.

5.2.2 Sewage Treatment Plant

The objectives of the WPCP model were:

i) to establish the impact of additional flows upon plant performance, where the additional flows originate from such combined sewer overflow (CSO) control strategies as storage and treatment (storage within the sewer system, treatment at the Woodward Avenue WPCP);

ii) to calculate the relative changes in loadings to Hamilton Harbour due to various control strategies;

iii) to evaluate the additional WPCP capacity required for such control strategies as storage and treatment

iv) to provide cost estimates for the WPCP portion of the wastewater collection - treatment system resulting from these control strategies.

5.2.3 Storm Sewer System

The objectives of the separated storm sewer system model were to determine constituent loadings under:

- existing conditions;
- future development conditions as specified in Figure 2.3, without remedial works; and
- future development conditions, with remedial works.

Unlike a combined sewer system analysis, however, where loadings are a function of CSO volumes and hence peak flows, a loadings analysis involving separated storm sewer areas need only compute runoff volumes. Therefore, a hydrograph based model was not required for the analysis. Rather, a spreadsheet based water quantity/quality model was used which was capable of modelling remedial works.

5.3 Overview of Selected Models

This section provides a brief overview on the features and limitations of the models selected for use in the PCP study.

5.3.1 Stormwater Management Model (SWMM)

The RUNOFF and TRANSPORT blocks of the SWMM were used to model the Woodward Avenue WPCP (combined) drainage basin. Reasons for selecting this model are outlined below.

1. The model may be set up to run in either a continuous or design event mode.
2. Quantity and quality characteristics of sanitary flows may be modelled.
3. Water quality constituents may be modelled for stormwater runoff.

4. Flow regulators may be modelled.
5. Storage facilities may be simulated.
6. SWMM has been successfully applied to similar studies, and publications detailing its application exist to aid the study team.

Although the RUNOFF and TRANSPORT blocks are powerful modelling tools they do have certain limitations which must be considered in this study. The most noticeable limitations are within the TRANSPORT block, namely:

- no more than 200 elements (i.e. pipes, manholes) can be modelled in one application of the model; and
- sewer surcharging is not modelled (i.e. all flow is uni-directional).

In this study, these limitations will restrict the size and complexity of the model (i.e. only the major sewer system elements could be modelled).

5.3.2 Overview of WPCP Models

Two different models were selected to analyze the impacts due to loadings to the WPCP. Basically the models assessed:

- WPCP performance; and
- costs of impact mitigation.

Knowledge of WPCP performance, firstly under steady state and secondly under non-steady state (CSO) conditions, serves as the basis for deciding on improvements to the WPCP. Having decided on improvements, the costs associated with these improvements can be established.

A process model called GPS was selected to evaluate process alterations due to

- increased dry weather flows
- storm related impacts.

A cost evaluation code, CAPDET was used to assist in evaluating the costs of these improvements.

The codes are described below in Section 5.4.3.

5.3.3 Stormwater Spreadsheet Model

The stormwater spreadsheet model used in this analysis estimates runoff volumes using:

- daily rainfall depths;
- depression storage losses for pervious and impervious areas; and
- volumetric runoff coefficients pervious and impervious surfaces.

Loadings were computed by combining the runoff volumes with event mean concentrations (EMC). Section 5.5.2 provides an overview of the EMC approach to water quality modelling.

5.4 Development of PCP Study Models

5.4.1 Combined Sewer System Model Development

Development Guidelines

The combined sewer drainage basin covers approximately 54 km^2 and includes extensive interconnected pipe networks. It is not feasible, nor is it necessary to model every aspect of this drainage basin. Rather, a lumped modelling approach can be used, provided it is not so coarse that the major drainage characteristics of the basin are missed. The following guidelines were, therefore, devised to ensure that an accurate lumped model would be constructed.

1. The major drainage patterns should be clearly defined. This criteria suggests that local sewers may be excluded, and that only the most significant combined trunk sewers need be included.

2. Major flow regulators, which control the discharge of combined sewage from combined trunk sewers to the sanitary interceptor sewers, or from one sewershed to another, should be included.

3. Local flow regulators, which regulate combined sewer flows within the local sewer systems should be excluded.

4. All major storm relief/CSO outfall sewers, as well as the main branches of the western and Redhill Creek sanitary interceptors should be modelled.

Model Development

Model development commenced with a thorough review of the Region's neighbourhood sewer maps. Using the sewer maps, subcatchment boundaries were defined using pipe flow direction, and the location of the major flow regulators identified. Detailed plan and profile drawings for the major trunk sewers and overflow structures were then extracted from the Region's print library. A review of these detailed drawings established sewer parameters (e.g. pipe geometry, slopes, etc.) and the hydraulic characteristics of the major flow regulators. Since many flow regulator parameters (e.g. weir height, sluice gate position, etc.) remained unknown after plan/profile review, a field survey of the major flow regulators was initiated. In total, approximately 30 flow regulators were investigated. Regional staff at the Barton Street works yard and at the Woodward Avenue WPCP also provided useful information on the history, operation and maintenance of the major flow regulators.

A brief overview of the combined sewer system model follows. Model schematics are included at the back of this report.

Subcatchment Boundaries/Inlets

Subcatchment boundaries include only those areas which direct combined sewage to the WPCP. Each subcatchment is assumed to drain to a single point. Generally, subcatchment flows are assumed to drain to a combined trunk sewer at the subcatchment's outlet.

Sanitary flows from areas serviced by separated sewer systems are included in the model by inputting a representative dry weather flow at the outlet of the serviced area (e.g. the Ancaster/West Mountain sanitary inflow point in the Redhill Creek Sewer Schematic).

The model was also constructed with the assumption that stormwater runoff from industrial lands north of Burlington Street drains directly to the harbour. This assumption was verified by Region staff. Consequently, these lands were not included in the combined sewer system model. Rather, these lands are included in the stormwater model.

Modelled Sewer Segments

Generally, only combined trunk, sanitary interceptor, and CSO/storm outfall sewers were included in the sewershed model. This decision implies that local flow regulators are assumed to direct all sewage to combined trunk sewers rather than partitioning flows between the combined sewers and the upstream sections of the CSO outfalls. Therefore, all flows are routed through combined trunk sewers to major flow regulators which determine the quantities of water which are either conveyed to the WPCP, or diverted to CSO outfalls. The resulting sewer system clearly defines the major flow paths within the Region's extensive sewer system.

Automatic and WPCP-Controlled Flow Regulators

The locations of the major flow regulators are included on the model schematics. These regulators consist of weirs, orifice plates, fixed and WPCP-controlled sluice gates which regulate combined sewage flows into the sanitary interceptors or storm outfalls, and the transfer of combined sewage between subcatchments and sewersheds.

Model Components

The resulting combined sewer system model was ultimately developed as three separate components, namely:

- West Hamilton and the Central Business District;
- Redhill Creek; and
- the Woodward Avenue WPCP wet well.

The decision to separate the combined sewer system model into three components was made to:

i) limit the total number of elements (e.g. pipes, flow regulators and manholes)to less than 200 (the maximum allowed by the TRANSPORT block): and
ii) save execution time by allowing the West Hamilton/Central Business District model, where 21 of the 23 modelled overflows exist, to be run independently.

Briefly, for any desired analysis, the two subcatchment models are run independently, the resulting inflows to the WPCP are stored, and then the WPCP model combines the inflows to determine treated and bypassed sewage volumes. The components of the models are summarized in Table 5.1.

TABLE 5.3

SWMM4 PARAMETER VALUES

	Value For	
Parameter	Lands Below Escarpment	Strathearne Model
Percent Impervious	51.0	54.0
Dry Weather Flow Rate (l/s/imp ha)	1.06	1.38
Ground Slope (%)	1.00	1.20
Flow Regulator Overflow Threshold (l/s/imp ha)	8.75	7.90

Note: All other parameters (flow lengths excluded) are common parameters and are listed in Table 5.5.

The next stage in developing the representative CSO catchment model was to determine if an existing catchment closely represented typical conditions. This review indicated the monitored Strathearne Avenue sewershed represented overall sewershed conditions (Table 5.3). Furthermore, the simulated runoff response for this catchment will be realistic since the runoff response of this area will be calibrated and validated with the flow and rainfall data presented in Section 4. The drainage area to the Strathearne Avenue outfall will, therefore, be used to represent typical conditions for a CSO outfall.

The entire drainage area for the Strathearne Avenue outfall includes the Lower Strathearne Upper Strathearne, Malta, Fairfield, Central and Adair catchments. Figure 88143–B–1 in the back of the report illustrates the area covered by the representative combined sewershed.

5.4.3 Model Development for Woodward WPCP

Prior to developing a model for the WPCP the study team reviewed:

1. Previous studies of the impact of CSO diversion on wastewater treatment plants.
2. Previous reports on the Woodward Avenue WPCP.

TABLE 5.1

COMBINED SEWER SYSTEM MODEL COMPONENTS

a) General Details

Area	Total Drainage Area (km^2)	Average Percent Impervious	Number of Sub–Catchments	Number of Sewer Elements	Number of CSO Outalls Modelled
Chedoke & Central Business District	40.4	51.4	37	162	21
Redhill Creek	12.8	44.1	9	44	2
STP Wet Well	0	0	0	9	1

b) Flow Regulator Details

Type of Flow Regulator	Number Modelled
1. Diverts flow to either the STP or a CSO Outfall	19
2. Diverts flow between combined sewershed or trunk sewer	15
TOTAL	34

Minor CSO Outfalls Not Modelled

Although the combined sewer system model includes all the main CSO outfalls, numerous small (i.e. local) CSO outfalls were excluded to simplify the analysis and to reduce the number of modelled sewer system elements. An overview of these outfalls is included in Table 5.2.

Simplifications to the pipe network upstream of the Ferguson outfall were also made which reduced the likelihood of modelling CSO's through the outfall. These simplifications tend to direct CSO volumes for the Ferguson outfall to the Wellington Outfall. Consequently, CSO volumes and frequencies for the Ferguson Outfall may not be realistic, and readers should refer to CSO characteristics at the Wellington Outfall. Storage facilities for the Wellington and Ferguson Outfalls will, therefore, assumed to be lumped.

5.4.2 Representative CSO Catchment Model

This model was used to conduct a detailed assessment of CSO characteristics, constituent loadings, and remedial works. The model was constructed because although the Region's combined sewer system is extensive, it is also highly repetitive. That is, each major combined sewershed:

- has one CSO outfall;
- has one or more flow regulators;
- usually has three or more land uses; and
- has a limited quantity of sewage which may be conveyed to the sanitary interceptor.

Therefore, since each combined sewershed has similar characteristics, it is possible to determine the response of a representative combined sewershed to rainfall and then to extrapolate the results over the entire combined sewershed. The correct application of this type of analysis requires that the selected sewershed by representative of the study area. Consequently, the study team reviewed the study area and determined:

1. The majority of the CSO's originate from the lands below the escarpment.

2. Areas above the escarpment are already serviced by the Greenhill CSO storage facility.

Consequently, only the hydrologic and hydraulic properties of the lands below the escarpment will be used to ascertain typical combined sewershed characteristics. Table 5.3 lists the average values for the hydrologic parameters for lands below the escarpment.

TABLE 5.2

MINOR CSO OUTFALLS

NOT INCLUDED IN SWMM4 MODEL OR CSO ANALYSIS

Location	Receiving Water Body	Outfall Diameter (mm)	Comments
Simcoe Street West (Sewer Map G-6)	Hamilton Harbour	375 and 675	Drainage area and CSO characteristics were lumped with the Hess catchment. CSO's are assumed to be stored in any facility proposed for the Hess outfall. Flows in excess of the pumping station forcemain capacity are lumped with Hess outfall CSO's.
Heath Street (Sewer Map Q-8)	Redhill Creek	750/900	Drainage area and CSO characteristics were lumped with the Melvin subcatchment. CSO's are assumed to be stored in any facility proposed for the Melvin outfall.
Studholm/Aberdean (Sewer Map E-11)	Chedoke Creek	450	Drainage Area = 1.4 ha (estimated) Percent Impervious = 45 Estimated offline CSO tank volume - for a 4 CSO events /year control strategy: 350 m^3 - for a 1 CSO events/year control strategy: 500 m^3 Detailed site investigation required to ascertain actual drainage area.
Glenside Avenue (Sewer Map E-12)	Chedoke Creek	750	Drainage Area = 15.3 ha Percent Impervious = 45 Estimated offline CSO tank volume - for a 4 CSO events/year control strategy: 1,750 m^3 - for a 1 CSO events/year control strategy: 2,450 m^3

An overview of these items follows.

Previous Studies of Impact of CSO Diversion on Wastewater Treatment Plants

There have been a limited number of studies which have rigorously evaluated the impact of CSO diversion upon WPCP performance using computer simulation. Several have been carried out in other countries. The need for such studies have lead the United States Environmental Protection Agency and others to develop generic micro computer based programs for simulating the performance of plants, and for estimating the associated capital, operation and maintenance costs (e.g. AXIAL, PROCESS, CAPDET). Such computer programs account for mass removal of suspended solids and BOD_5 across clarifiers, uptake of BOD into MLSS, aeration requirements, dissolved oxygen levels and flow balances. They require empirical (statistical) relationships for suspended solids across clarifiers to predict clarifier performances.

A good example of the use of such software is the evaluation of the impact of internal sewershed storage and subsequent diversion of the wastewater to the Humber Wastewater Treatment Plant. Internal storage within the collection system was provided to mitigate basement flooding and to control CSO discharges to the Humber River. The impact of diversion upon hydraulic capacity of primary and secondary unit and the performance of these units for control of BOD_5, SS and TP were evaluated. The costs of expansion to handle CSO diversions associated with a 1 year storms were then evaluated. The analysis is reasonable for a comparative evaluation of plant performances under different flows. The analysis suffers from attempting to relate the model calculations to absolute effluent concentration criteria when the empirical removal functions for suspended solids are quite variable. An inadequate plant data base for model calibration is another limitation.

In addition, similar software was used to evaluate treatment options for handling rainfall impacts upon the Main WPCP in Central Metropolitan Toronto.

Previous Studies on Woodward Avenue WPCP

There have been several previous studies on the Hamilton–Wentworth WPCP including:

i) Studies on the hydraulic capabilities and limitation of the WPCP, (e.g., Pollutech (1979, 1980 a, b, c, 1981, 1983)

ii) Characterization of sludge handling and incineration capabilities at the WPCP (e.g., Proctor and Redfern, 1987)

iii) Influent characterization and evaluation of removals of metals and trace organics by CANVIRO. (This study also included an attempt to relate BOD removal to sludge age).

iv) Empirical modelling at a unit process level, of the partitioning of various metals and organics into various streams (effluent liquid stream, solids stream, anaerobic digester gas, incinerator gases, incinerator ash, landfilled solids) at the Hamilton WPCP (BEAK; 1986).

v) Characterization of volatilization of a few metals and organics by BEAK and Goodfellow in a worker–safety oriented study.

vi) Application of mass balance based, performance models to the WPCP by a fourth year chemical engineering design course at McMaster (Patry, Tsezos, Rymak) for a design course.

vii) Development of mass balance study (Snodgrass, 1981) which evaluated the relative impact of the WPCP, CSO's and storm sewer discharges on harbour water quality and which evaluated plant performance in a general sense.

As well, there is considerable practical experience in running the plant for achieving control of organic carbon oxidation and removal and understanding the plants limitations in achieving nitrification.

These studies provided the data for examining hydraulic limitations in the plant. Performance evaluation of the plant (Tsezos et al.) using simulation modelling was also quite helpful; however, it was steady state. This necessitated development of a non steady state model to understand the effects of transients on plant performance.

Procedures Used to Develop WPCP Models

WPCP Process Model

The performance of the Woodward Ave. WPCP was analyzed through the use of a WPCP model (GPS_W). GPS_W is a computer package in use at McMaster University for the analysis (steady and

non–steady state) of wastewater treatment processes. This package was initially developed by the McMaster Group under contract to Environment Canada. It was made available to this project, and was tested with full scale, Woodward Avenue WPCP data.

Simplified Plant Layout

In order to keep the mathematical model executable within reasonable CPU time some simplifications were applied to the complex layout of the real plant. One of the most obvious possibilities to simplify the model is to simulate parallel technological units as one unit, with the combined volume and surface of the individual units. This approach is feasible to the extent that as there is reasonable evidence that the loading of the parallel units does not differ extensively from each other. In this case, this means that the primary clarifiers, the aerators and settlers within one process line can be modelled as single units, and that the north and south process line have to be dealt with separately, due to the different loading they receive.

Another simplification was to omit all the technological processes not directly associated with the performance of the activated sludge process (anaerobic digestion, incineration, etc.). Some of these processes have an impact on the activated sludge process by contributing to the load of the plant (digester supernatant, sludge filtrate, scrubber water). This load was incorporated into the influent of the plant as a specified input.

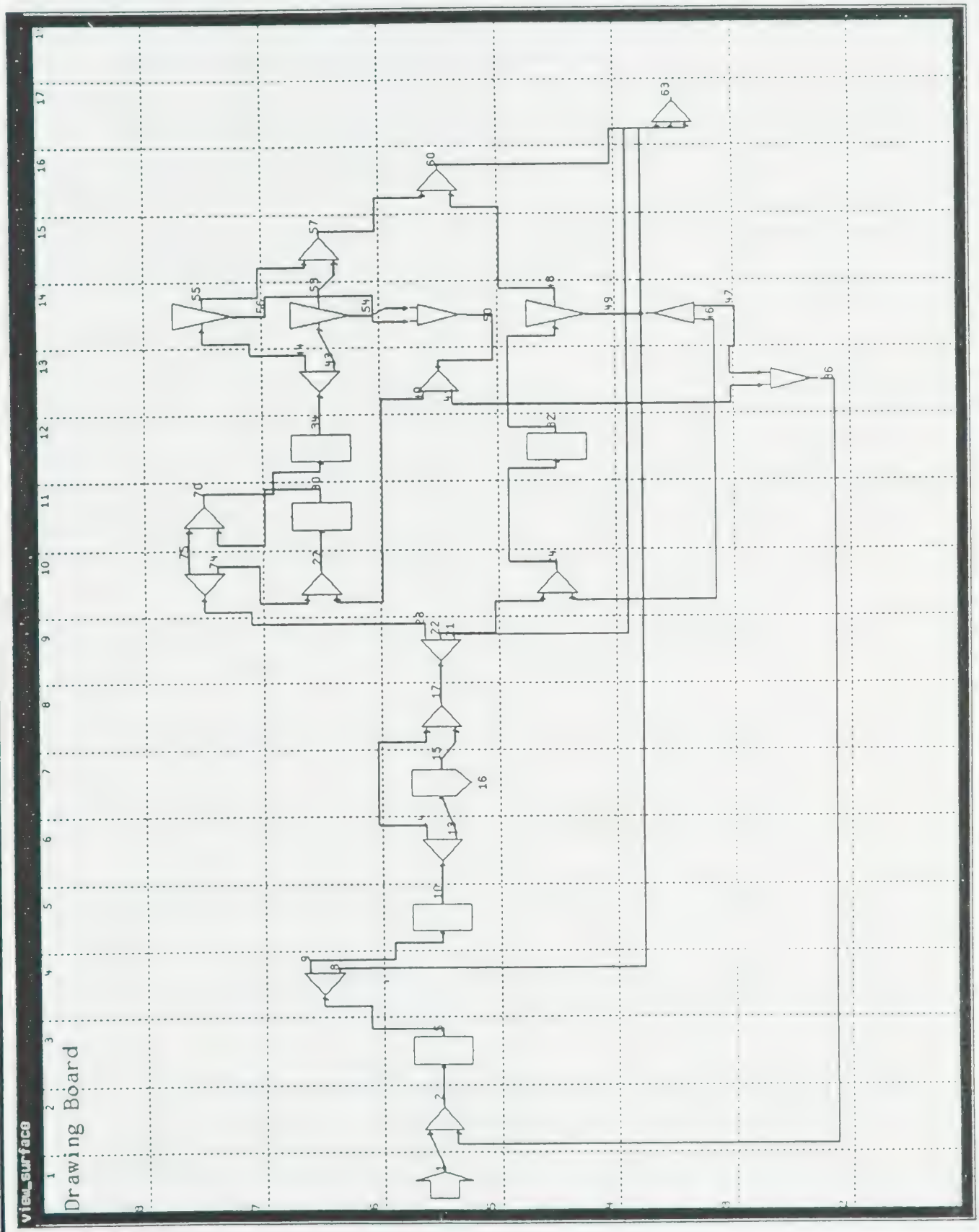
The layout of the simplified plant, which was used as the basis for the WPCP model, is shown in Fig.5.1. The simplified flow schematic shows flows entering the head of the plant from the sewer system (line 1) and sludge recycle (line 36). The primary portion of the plant is represented as line 6 to line 17 with two by passes (line 8: complete plant by pass; line 14: primary by pass). The secondary portion of the plant is represented as Line 17 to line 60. The secondary portion is divided into two sections – the old side of the plant and the new side of the plant.

The WPCP model allows the following calculations.

a) After accounting for secondary sludge wastage (which is measured as a portion of the influent plant load due to the wastage line joining the interceptor sewer before influent plant samples are taken), what is the sewer system loading to the plant. This provided data for calibrating the water quality portion of the SWMM model.

Figure 5·1

SIMPLIFIED LAYOUT OF THE HAMILTON WPCP



b) What is the probable present loading to the harbour from the WPCP due to a variety of rainfall events. This provides estimates of by pass loadings. The present data base records length of time of by passing different sections of the plant, but does not readily account for differences in concentration when flow variations occur.

c) What is the change in the WPCP loadings to the harbour due to various CSO control strategies or due to the addition of major primary or secondary units to the plant.

The WPCP model was formulated to factor out the effect of Waste Activated sludge return upon plant loadings and to readily allow other calculations which were useful to this study.

2. WPCP Cost Model

After determining the appropriate plant modifications, the CAPDET code was used to cost these modifications. CAPDET is a Computer Assisted Procedure for the Design and Evaluation of wastewater Treatment Systems. This computer program is a planning tool which permits the evaluation of costs of alternative treatment unit sizes. The Canadian version which reflects local cost conditions to the degree currently possibly, was used.

The CAPDET program was coded to suit the Woodward Avenue WPCP. Different trials were made to allow evaluation of different sizing scenarios.

5.4.4 Stormwater Spreadsheet Model Development

The stormwater spreadsheet model was developed using the following procedures:

1. Drainage boundaries along each major watercourse were determined using 1:50,000 topographic mapping of the area.

2. The extent of existing and future development within each drainage basin is based on the 1:50,000 Region map "The Regional Development Pattern (1989)".

3. Runoff coefficients for developed areas were computed using air photographs, land use maps and volumetric runoff coefficients from both the analysis of the combined sewer system and from areas analyzed in other studies with similar characteristics.

4. Runoff coefficients for rural areas slated for development were ascertained by reviewing soil types in the area, and the volumetric runoff coefficients of gauged watercourses within the Region.

5.5 Rainfall and Water Quality Data For Modelling

5.5.1 Rainfall Data

Introduction

The next step in the study was to select representative rainfall data for analysis. Mount Hope Airport rainfall data was selected to represent typical rainfall patterns as it is located in the Hamilton area, has 19 years of records, and has been found by other researchers to be more representative of rainfall patterns in the Hamilton area than the Royal Botanical Gardens rainfall gauge (Mark Robinson, 1989). The following sections outline the selection of rainfall data for analysis.

Selection of Rainfall Time Series

The Terms of Reference suggest that any system analysis need only be conducted for a typical year. However, experience with previous studies have indicated to the study team that, with a little more effort, drainage basin characteristics can be assessed for 3 years – dry, wet and typical. Such analysis, for example, provides more insight as to the range of CSO frequencies/volumes that can be expected in any particular year. Furthermore, a more thorough analysis of remedial works can be performed with three years of data available for analysis. For these reasons, a dry, wet and a typical year was selected for analysis.

Two parameters were used to classify the rainfall characteristics of any particular year: total rainfall and the distribution of rainfall events from 1 mm to 20 mm, in 4 mm increments. These parameters are both simple to compute, and provide a good indication of rainfall characteristics.

Selection of Wet and Dry Years

Figure 5.2 summarizes the total rainfall depths observed at the Mount Hope Airport from 1971 through 1989. A review of this figure suggests that appropriate dry and wet years for CSO analyses are 1971 and 1981, respectively. These years were, therefore, selected for system analysis.

Selection of a Typical Year

The typical year chosen for analysis should have:

- a rainfall depth near the long term average depth;
- a distribution of rainfall events representative of a typical year; and
- a good rainfall database.

The 19 year average rainfall depth from May 1 to October 31 is approximately 490 mm (Figure 5.2). A further review of Figure 5.1 indicates that the Region received an average amount of rainfall in 1975, and that 1989 was fairly representative of long term trends.

To further investigate the rainfall patterns in 1975 and 1989, the distribution of rainfall events in each year was plotted against the long term averages (Figure 5.3). This analysis indicated that both years are representative of long term conditions, but that 1989 had numerous events in 4.1 – 8 mm range.

Next, the advantages and disadvantages of using either 1975 or 1989 as the typical year for analysis were considered. This review indicated that in 1975, very little detailed rainfall data is available in the City and no WPCP data is available. Conversely, in 1989, numerous rainfall gauges were operational throughout the City, and detailed flow monitoring and WPCP flow records are available. Therefore, 1989 was selected as the typical year for analysis since a more complete database is available to complement airport rainfall records.

Figure 5.4 illustrates the daily rainfall depths during the adopted dry, typical and wet years. The most noticeable trends from Figure 5.4 are:

- the dry, typical and wet years have approximately 1, 5 and 7 events over 30 mm in depth, respectively;
- the dry and typical years have few, if any, consecutive days with large (i.e. greater than 10 mm) rainfall depths, whereas the wet year has several occurrences; and
- interevent times tend to be shortest in the wet year.

These observations suggest that runoff response characteristics, such as combined sewer overflow volume, may vary substantially from year to year.

FIGURE 5.2: TOTAL RAINFALL FROM MAY TO OCTOBER AT MOUNT HOPE AIRPORT

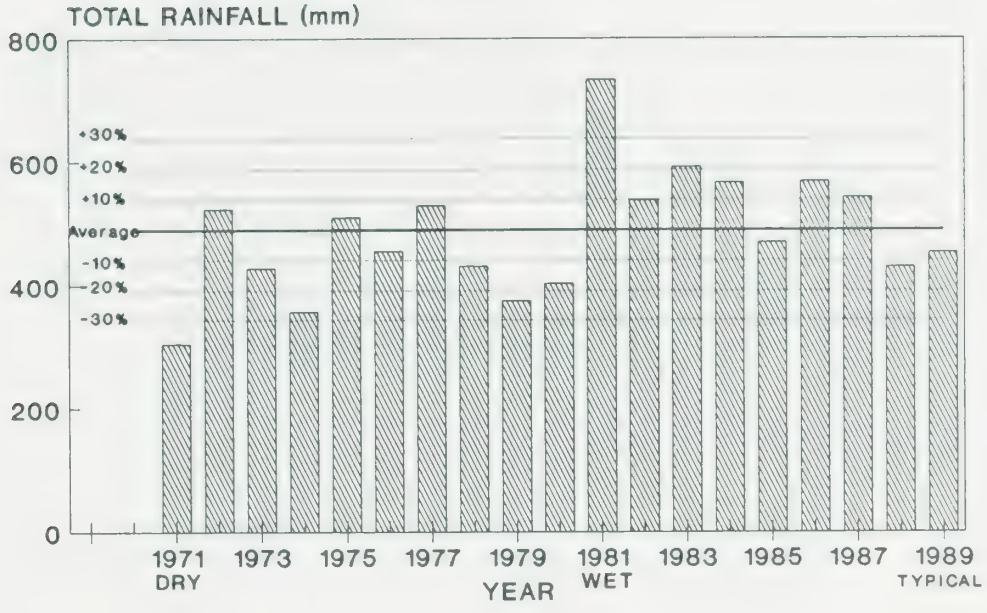


FIGURE 5.3: DISTRIBUTION OF MAY TO OCTOBER RAINFALL EVENTS

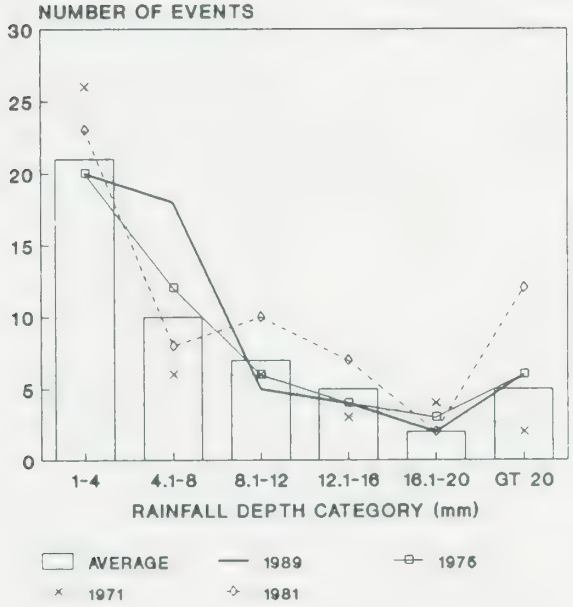
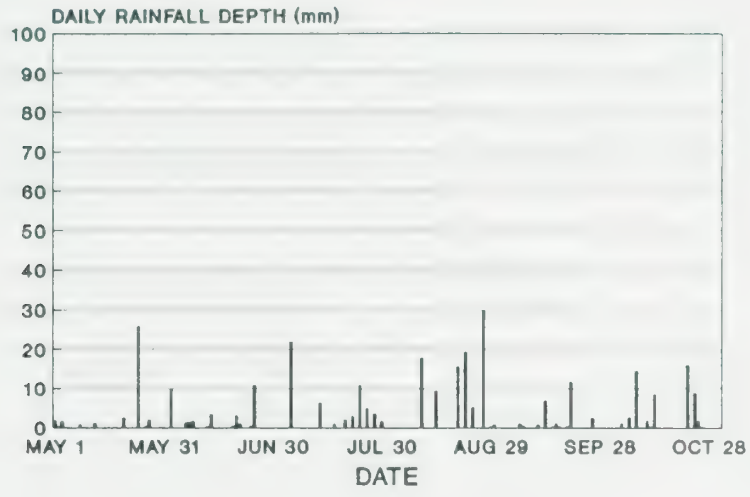
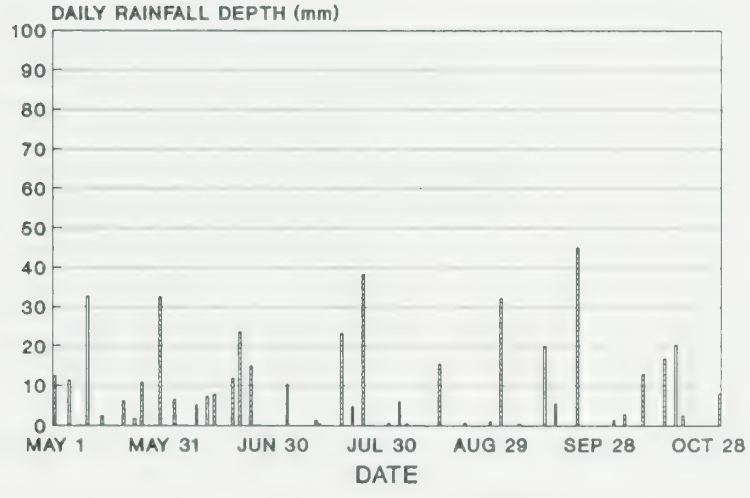


FIGURE 5.4: DAILY RAINFALL DEPTHS AT MOUNT HOPE AIRPORT DURING DRY, TYPICAL AND WET YEARS



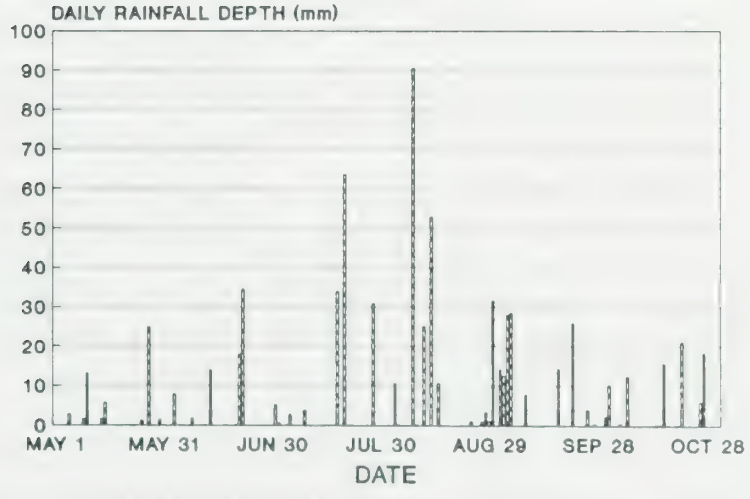
DRY YEAR
1971

TOTAL RAINFALL
294 mm



TYPICAL YEAR
1989

TOTAL RAINFALL
454 mm



WET YEAR
1981

TOTAL RAINFALL
706 mm

NOTE: 19-YEAR AVERAGE (MAY 1 TO OCT 31) = 490 mm

Design Rainfall Events

Design rainfall events were also required to:

- ascertain system response for typical (i.e. 5 mm) to severe (i.e. 35 mm) rainfall conditions; and
- provide rainfall input to size CSO detention facilities.

Selection of the design rainfall events commenced with a review of the proposed levels of CSO control, namely, 4 events per year and 1 event per year (see Section 7). Nineteen years of rainfall data were then reviewed to ascertain rainfall depths which, on the average, were exceeded 4 and 1 times per year. This review yielded rainfall depths of approximately 25 and 35 mm, respectively.

Next, an appropriate design rainfall hyetograph was required to distribute the selected rainfall depths. Consideration was first given to the widely used 1–hour AES design storm. However, 25 and 35 mm of rainfall distributed over 1 hour is equivalent to return periods of 2 and 10 years, respectively. These return periods represent much more critical conditions than required. Consequently, a 2–hour design storm hyetograph was adopted from the 1–hour AES design storm to distribute the proposed design rainfall depths.

5.5.2 Selection of Water Quality Parameters

Combined sewer overflows, stormwater discharges or WPCP bypasses or effluents may impact receiving bodies of water by discharging excessive concentrations or pollutant loadings (mass x concentrations) to a receiving body of water. The excessive concentrations or loadings may result in problems due to:

- destruction of aquatic habitat
- impact on recreational uses
- eutrophication of the water body
- impact on resident fishery
- visual impact (i.e. debris, floatables, colour of water)
- impact on drinking water sources.

Recent studies have identified over 150 water quality parameters in stormwater discharges and CSO's. The assessment of the impact of such a large number of parameters for a study of this nature would be onerous. As such, a reduced list of parameters were considered for this study.

The selection of the key parameters to be assessed in this study was based on several considerations. These include:

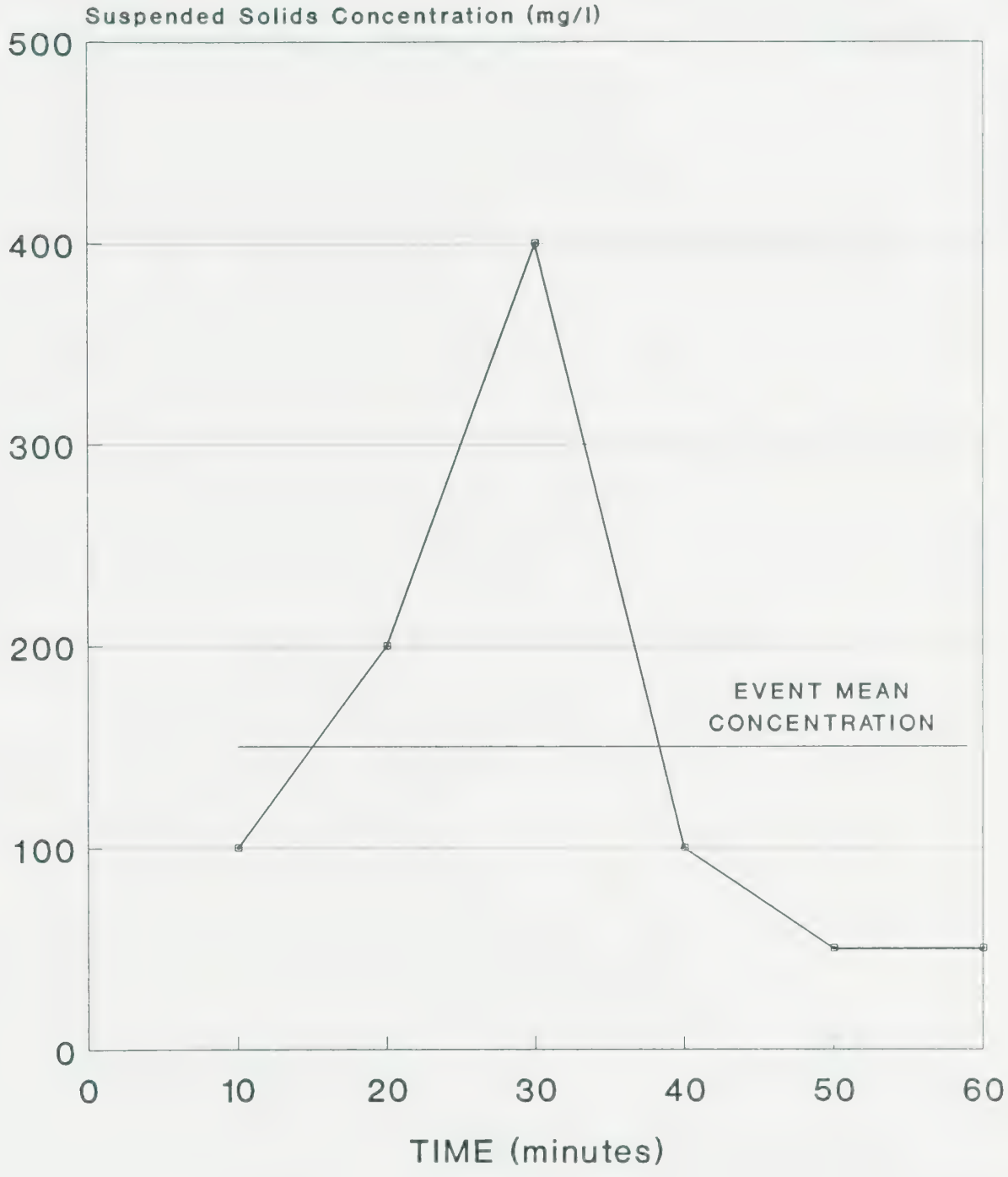
- pollutants which were identified as "priority pollutants" in the Hamilton Harbour Remedial Action Plan (HHRAP)
- the Terms of Reference for this study
- discussions with the Technical Committee
- existing and proposed uses within the receiving bodies of water (covered in part by the HHRAP)
- the traditional parameters which are generally considered due to high concentrations within the discharges
- the determination of a reasonable number of parameters to be assessed.

Based on the above considerations a total of 10 water quality parameters were selected for determination of the pollutant loading. The selected parameters are listed below`.

- Zinc; (Zn)
- Fecal Coliform; (FC)
- Total Phosphorus; (TP)
- Suspended Solids; (SS)
- Ammonia; (NH_3)
- Total Kjeldahl Nitrogen; (TKN)
- Dissolved Solids; (DS)
- Copper; (Cu)
- Lead; (Pb)
- Biological Oxygen Demand; (BOD_5)

The approach used to assess the loading of each parameter to the receiving waters centred on the use of an Event Mean Concentration (EMC) for each constituent. The EMC is defined as the average concentration of a constituent over a period of time. Figure 5.5 illustrates this concept by showing

FIGURE 5.5: HYPOTHETICAL POLLUTOGRAPH AND RESULTING EVENT MEAN CONCENTRATION



the resulting EMC for an event in conjunction with a hypothetical pollutograph. Constituent loadings are computed using EMC's by the formula:

Constituent Loading	=	Discharge Volume	X	EMC of Constituent in Discharge

The EMC approach to constituent loadings was adapted in this study for the reasons outlined below.

1. Several studies (NURP, 1984 and Peterborough, 1989) have recommended using EMC's to define the response of pollutant discharges over time.

2. Studies have suggested:
   - EMC's for any particular event are relatively independent of the volume of the event; and
   - constituent loadings from stormwater are generally proportional to runoff volume.

3. The buildup/washoff approach to constituent loadings was not feasible as sufficient data was not available to calibrate/validate the buildup/washoff characteristics of the Hamilton system. Some time history concentration data was available from previous studies conducted at McMaster University. The majority of the samples were, however, taken within watercourses and did not seem to reflect characteristic values for either stormwater or CSO's. Furthermore, the use of an uncalibrated buildup/washoff quality model has been shown to produce questionable results (SWMM4 manual, 1989).

4. EMC data has been shown to be transferrable from one geographic location to another.

The general approach taken to apply EMC's was to make use of the extension NURP data base, and to compliment this data with local data. A brief review of these studies follows.

NURP and Related Studies

The United States Environmental Protection Agency completed a National Urban Runoff Program to define water quality constituents. The program, which involved monitoring in over 40 sites throughout the United States, is generally acknowledged to have produced the most comprehensive database in North America.

Figure 5.6 illustrates the relationship between various EMC's for a series of events. The results of the United States Environmental Protection study as well as several studies carried out within Ontario suggest that the Average of the Event Mean Concentration can be reasonably determined if a minimum of 10 to 12 events are monitored. Furthermore, the range of EMC's may be characterized by the average value and the standard deviation.

Two other conclusions were drawn from the NURP study. These are:

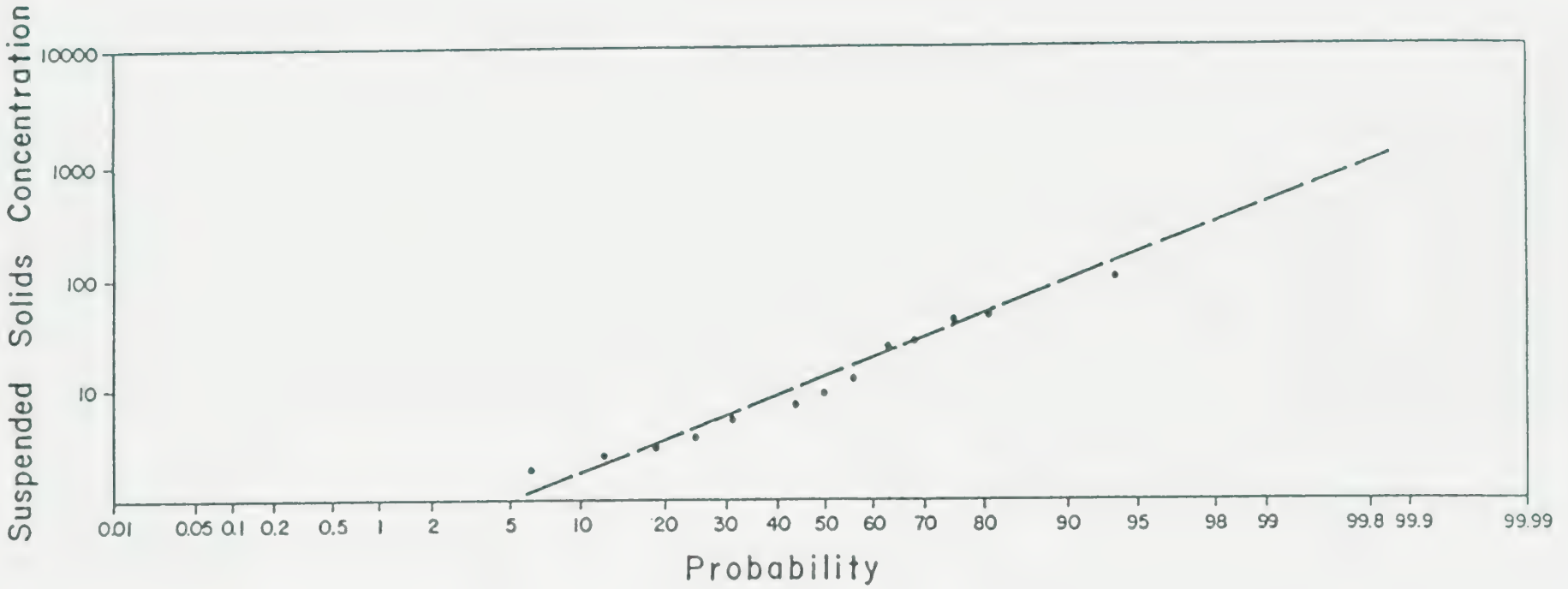
- the Average Event Mean Concentration (AEMC) does not vary significantly for various urban land uses (i.e. commercial vs. industrial vs. residential)
- the Average Event Mean Concentration is reasonably uniform for different geographical regions.

The above findings were used to establish Average Event Mean Concentrations for this study for each of the 10 water quality parameters. Values for the AEMC were selected based on the findings from the NURP and other Canadian monitoring programs as water quality monitoring data was not available for this study. Monitoring data from the Woodward WPCP was used for defining sanitary sewage values. WPCP effluent concentrations were based on monitoring data from the plant and concentrations specified in the plant's Certificate of Approval.

The Average Event Mean Concentrations for sanitary and stormwater sewage and WPCP effluent for each of the 10 selected parameters are shown in Table 5.4. The Event Mean Concentration for CSO's will be defined based on the relative volume of stormwater vs. sanitary sewage discharges (see Section 6).

Limitation of the EMC Approach to Constituent Loadings

Although using the EMC approach to estimate constituent loadings for a planning level study is appropriate, it is not without its limitations. One shortfall of the EMC approach is it does not consider that watershed and atmospheric activities can only generate a finite quantity of pollutants for transport within a given time period. Therefore, for example, while constituent loadings for an EMC based approach may get infinitely large with increasing runoff volumes, the accuracy of the analysis may decrease with runoff volume since watershed/atmospheric processes may not be able to generate the quantity of pollutants estimated. In addition, numerous processes/conditions are not accounted for by the EMC approach. An overview of these processes/conditions is included in Appendix I. In



Illustrative Figure of Event Mean Concentration for 13 Events

Figure 5-6

TABLE 5.4

EMC DATA FOR SANITARY SEWAGE,

WPCP EFFLUENT, STORMWATER RUNOFF

Parameter	Sanitary Sewage EMC (mg/l)	WPCP Effluent EMC (mg/l)	Stormwater Runoff EMC (mg/l) Urban Areas	Stormwater Runoff EMC (mg/l) Rural Areas
SS	302	20	100	150
TP	6.0	0.4	0.33	0.46
Ammonia	14	6	0.3	1.2
TKN	27.4	8.8	1.35	2.84
Zinc	2.6	0.009	0.16	0.006
FC (#/100ml)	1×10^8	100	25,000	1,000
DS	430	430	350	390
BOD_5	129	8	9.00	2.0
Lead	0.071	0.006	0.14	0.004
Copper	0.10	0.018	0.034	0.004

NOTES:

1. EMC data was obtained through:

   Analysis of STP influent and effluent.

   NURP or Woodward-Clyde Reports.

   Observed stormwater flows from urban areas throughout Ontario.

   Rural stormwater flows observed in the Picton area.

   MOE's "Technical Guidelines for Preparing a Pollution Control Plan.

   A review of the Woodward Avenue WPCP's Certificate of Approval.

summary, the constituent loadings presented in the report are intended only for pollution control planning purposes.

5.5.3 Development of Data Base for WPCP Simulation Studies

Methods

Nine months of data were extracted, from the 1989 Woodward Avenue operating Record for influent, primary effluent and final effluent quality, as well as other important operational data (flows, MLSS, wastage, bypasses). This constituted about 8000 separate pieces of data. Several hours were also spent interviewing the plant supervisor and operating personnel about their experience. This was used to estimate some of the parameters (e.g. bypass flow rates), as no direct measurements were available.

In order to gain a better insight into the dynamics of the plant, four storm events were also sampled plus one normal dry weather flow day to estimate influent concentrations. Special attention was given to the performance of the primary and final settlers. As well, an experimental overloading of the most sensitive part of the plant was also carried out. The final clarifiers on the old (north) side, were shock loaded to determine the impact of short term, high value disturbances, since the storm event sampling indicated that they were the bottleneck to the operation of the plant.

All the experimental data was used for the calibration and verification of a dynamic mathematical model. This model was adapted specifically to fit the Hamilton WPCP, and to handle, although in somewhat simplified manner, the full process train of the plant excluding the sludge handling train. The verified model was then used to simulate certain events in order to draw conclusions and allow predictions regarding the operation of the plant. Both long term and dynamic control policies were investigated.

Plant Record Extraction, Operator Interviews

Operating data were extracted from the plant records for the period January 1 – September 30, 1989. The data included influent, primary effluent and final effluent quality (suspended solids, BOD, TKN or ammonium-N), as well as operational data for the hydraulics of the plant (influent load, bypasses, recycles, wastage flows and MLSS in the aeration tanks). The data were averaged on a monthly basis and annual basis.

The plant has two major periods: a winter (cold weather) period from January till April, and a summer (warm weather) period from April to September. The main differences in the operating regimes include:

Parameter	January – April	April – September
Influent suspended solids, g/m^3	400 or above	250 – 400
Influent BOD_5, g/m^3	150 or above	approximately 100
Primary effluent solids, g/m^3	120 – 240	below 90
Primary effluent BOD_5, g/m^3	around 100	around 400
Dissolved oxygen, g/m^3	above 1	below 1

There is a much higher load during the winter months. However, the dissolved oxygen concentration is lower during summer. This is possibly due to the higher degree of partial nitrification during summer, maintained by elevated temperatures, which depletes the dissolved oxygen concentration in spite of the lower load.

The practical experiences collected from the plant operators, mostly from Robert Crane operations supervisor, can be summarized as follows:

1) The final clarifiers, in spite of their identical design on the north side operate differently. Clarifier #7 & 8 build up a sludge blanket easier and it is harder to prevent loss of biomass over the weir from these units, even with increased recycle. The explanation of this phenomenon is not known. It might be partially blocked recycle pipes.

2) The operating policy of the final clarifiers is to try to keep the sludge blanket as low as possible, usually around 30 cm (1 ft). This helps to minimize sludge washout during storms. The likelihood of sludge washout is also decreased by keeping low MLSS concentrations, particularly in the north side of the plant.

3) The north side of the plant nitrifies partially, but the south, does not. At the time of the measurements program (August–September, 1990), half of the south side aeration capacity was unused (two basins empty). At the same time, six of the eight north side aeration

basins were operated, with only five of the six rotors usually operating in each of them. This is 62% of the aeration capacity (5 times 6 rotors out of 6 times 8 = 30/48 = 0.625)

4) The south side final clarifiers are stable even during storms.

5) Although it might seem that more load should be allocated to the south side of the plant during storms (since this side of the plant has more settling capacity, it is not easy in reality to accomplish this due to the control device (gate) for load allocation.

6) The following Bypass policies are used if storms occur: They involve three different levels of flow

   i. Automatic bypass of primary settlers is carried out, if flow is greater than 386 MLD (85 MGD).

   ii. Secondary bypass (before the aeration tanks) is started if sludge washout from any settlers, usually on the north side, is not controllable. The volume of bypassed flow is not measured; only the length of the period of bypass is recorded. After consultation with the plant supervisor it was decided, that about 20% of the actual flow is bypassed.

   iii. Pumphouse bypass, i.e., bypassing the whole plant, is started if flow is over 130–140 MIGD (600 MLD). The present pipes and channels are unable to handle more hydraulic load on the plant, regardless of the degree of overload of the settlers.

Sampling of Dynamic Events

In order to acquire some insight into the dynamic behaviour of the plant, particularly the settlers, and to calibrate the dynamic model, a storm monitoring program was undertaken. The objective was to collect samples at three different locations in the plant (influent, primary effluent, final effluent) and the necessary operating data during different magnitudes of storm conditions. Five periods were sampled, labelled herein Event #1, #2, #3, #4 and #5.

Event #1: 405 MLD storm. The first event was a small, but steady storm (rain) on 4th of August, 1989 (peak flow rate: 405 MLD, 90 MIGD). To help determine the storm volume, first the average dry weather diurnal pattern was determined by averaging daily flow patterns for August and September days having no appreciable precipitation. Then the storm flow and the dry weather flow were plotted together and the storm flow volume determined from the difference.

The plant was able to handle this storm successfully in that no overload of the final clarifiers was observed. However, the primary clarifiers were bypassed for 3 hours. There was some increase in the sludge blanket level in the north settlers, however it was controlled by increasing the recycle ratio. The south settlers didn't show any sign of response in the form of elevated sludge blanket to the storm.

It was also determined, that the sludge blanket level within any one of the north settlers was uniform. This finding was not supported by subsequent investigations into bigger storms.

Event #2: "Clean influent". The influent to the plant contains the waste activated sludge (a.s.) at the point of sampling. There is no physical way to sample the influent itself before mixing with waste a.s. Consequently it was decided to shut the wastage down for a period and to determine the contribution of the wastage to the influent suspended solids, BOD, and TKN. The experiment was performed on 10th August 1989 by shutting down the incinerator and filtrate water inputs for that day, and by cutting off sludge wastage.

There was some variation in the sludge blanket of the north settlers during the day, generally the level went up by about 20 – 60 cm during the day. This was regarded as the normal diurnal variation in the sludge blanket height. It was also found, that the arm of the settler's raking mechanism induces formation of a "bulge" in the sludge blanket around in front of itself which can reach 40 cm (1 foot) in height.

Event #3: 610 MLD storm. A 15th of August storm caused a peak flow of 614 MLD (135 MIGD). The settlers became severely overloaded. The shift foreman decided to start the secondary bypass to prevent excessive amounts of sludge washing out from the north settlers. Recycle was gradually increased to the maximum (about 85%). The south settlers did not build up any appreciable sludge blanket during the storm.

Event #4: 540 MLD storm,. On 14th September one settler (#5) was out of service during this event (590 MLD; 130 MIGD), and the parallel settler (#1) was continuously overflowing sludge during the 9 hour sampling program. The other settlers were handling the load well, after some initial overflow. Recycle was set to a maximum of 500 l/s in each settlers and secondary bypass was also applied. The initial sludge blanket height had been lowered to the minimum possible, since the storm was "expected". The south settlers did not react to the storm.

Event #5: Hugo (720 MLD). Hurricane "Hugo" downgraded to a tropical storm hit Hamilton on 22nd of Sept. The hydraulic load of the plant increased briefly to a peak flow rate of 720 MLD (160 MIGD) and the plant bypass had to be started to prevent flooding of the plant. The maximum influent load to the plant was kept at approximately 600 MLD by allowing bypassing. Recycle was increased to 65%. Settler #1 was out of service during the storm.

The north settlers could not prevent washout of solids during the sustained high load for approximately for 10 hours since the load was higher than 455 MLD (100 MIGD). Solids washed out even when there was no sludge blanket at the bottom of the settlers. Some of the settlers, when visually examined, alternated between a "full" and "empty" state several times during the storm.

Experimental Overloading

Experimental overloading of the most sensitive part of the plant was carried out. The final clarifiers on the old (north) side, being considered the bottleneck to the operation of the plant, were shock loaded to determine the impact of short term, high value disturbances. The overload experiment clearly showed the non-ideal behaviour of settlers. After only five minutes, the first sludge clouds appeared at the weirs, and within one hour complete sludge washout started. At the same time, the sludge blanket in other parts of the settler was still 1.5 – 2 m from the top. In other words, under sudden load increases only part of the final settlers is effective due to the hydraulic shortcut developing in the weirs' area. The simulation of this effect proved to be a major challenge undertaken in this study.

Analysis of Storms

The plant performance is reflected in the events sampled during the study as follows:

Period No.	Date	Rainfall	Peak Flow Rate	Bypassing/Comments
1	Aug. 4	5.7 mm	405 MLD	• Handled Storm with only minor bypassing of primary clarifiers
3	Aug. 15	15 mm	610 MLD	• Final Clarifiers Overloaded • Secondary Bypass Initiated
4	Sept. 14	19 mm	590 MLD	• North side – One clarifier bypassed • South side settlers – worked well
5 (Hugo)	Sept. 22	25 – 40 mm (measured)	720 MLD	• Plant bypassed; actual influent flow rate not measured • North side settlers washed out for 10 hrs.

These findings are consistent with the observations made from the plant records which indicate that:

- the plant can handle small storms (less than 4 mm which occur 45% of the time) and could accept additional overflow volumes if regulators were set differently for rainfall events of this size
- internal plant bypasses occur routinely for medium sized storms (5–11 mm)
- prolonged internal Woodward Ave. plant bypassing occurs for large sized storms (15–40 mm); whole plant bypassing also occurs.

The data also suggest that a transition zone may occur in the 4–6 mm size of rainfall dependent upon the rainfall dynamics (intensity; its location in the city; and the areal extent of coverage) with the present plant being capable of handling some storms and not others.

The event monitoring data for the plant very clearly show:

- the primary clarifiers are not operated to handle any more flow than 360 – 380 MLD, which corresponds roughly to a 4–6 mm storm dependent upon its dynamics;
- the present north side clarifiers begin to overflow (prompting secondary bypass) when the peak storm flow rate is in excess of the range of 500–600 MLD. The event data do not clearly define the actual value because it was for flow rates 600 MLD and above. The plant operating criteria are approximately 540 MLD (long–term). This suggests that the range for keeping the present clarifier, operating safely is below the range of 500 – 540 MLD for peak flow rates.

5.6 Calibration and Validation of Combined Sewer System Model

5.6.1 General

SWMM4 is a deterministic modelling tool. Therefore, provided input parameters are representative of the study area, the model will closely simulate the hydrologic and hydraulic response of the study area. During calibration, the performance of the model was ascertained by a comparison of simulated with observed basin responses. Input parameters were gradually adjusted within acceptable ranges until the best fit between simulated and observed response was obtained. The parameters selected to compare simulated versus observed response included flow volumes, peak flows, and hydrograph timing. Each model was assumed to be calibrated once the simulated responses were ±20 percent of the observed responses.

Calibration/validation of the combined sewer system model commenced at upstream sections of the basin and proceeded downstream to the WPCP itself. The points below summarize the process.

1. Model calibration/validation commenced with the drainage basins upstream of the Strathearne/Vansitmart and Fennell/East 25th flow monitoring sites. These areas are representative of the study area and have detailed flow and rainfall data available for the 1989 summer.

2. Calibrated/validated parameters obtained from Step 1 were then extrapolated over the entire drainage basin.

3. Next the performance of the entire model was checked at:

    i) Queen/Barton (sanitary interceptor sewer flow monitoring site);
    ii) the Plymouth Street CSO outfall; and
    iii) WPCP influent channel.

The process outlined above allowed for detailed calibration of representative catchments and checks on model performance at key locations. Flow data from the Birch/Brant sanitary interceptor flow monitoring site could not be used in the procedure due to backwater, a hydraulic process which is not modelled by the SWMM4 TRANSPORT block.

5.6.2 Selection of Calibration/Validation Events

Numerous events for which accurate rainfall and flow data are available are necessary to calibrate/validate the sewershed model. Maalel and Huber (1984) concluded that model calibration can be simplified and enhanced by calibrating continuously to a series of events, rather than calibrating to numerous single events. If applicable, their procedures were adopted in this study since the model is to be used for continuous simulation.

The rainfall events in July 1989 were chosen for model calibration. July 1989 consisted of six rainfall events ranging from 2 to 22 mm of rainfall (see Table 4.2). These six events range from short duration/high intensity events to long duration/low intensity events, and provide a broad database for model calibration.

For model validation the events of September 1 (2 events), 8 and 14, 1989 were used. These events range from 2 to 25 mm of rainfall and constitute a good database for model validation.

The numerous rainfall events in June 1989 were not used for either model calibration or validation because rainfall data was not available at the Blessed Sacrament and the St. Anne's rainfall gauging stations until June 24, 1989.

5.6.3 Assessment of Catchment Parameters

Catchment parameters (eg. percent impervious, ground slope, etc.) were initially estimated using land use maps, air photos, and topographic maps. Overall, five general land use activities were identified: single-family residential; multiple-family residential; industrial; commercial; and parkland/open space.

One of the most important parameters to ascertain in this study was the extent to which roof downspouts were connected to the combined sewer system. Impervious drainage area increases substantially if roof downspouts are directly connected to the combined sewer system, resulting in quicker runoff response, larger runoff volumes, and more peaked flows. A field survey was, therefore, conducted to determine downspout placement. Downspouts which were observed to discharge below grade were assumed to be directly connected to the combined sewer system. This survey indicated that in excess of 90 percent of the downspouts in the Region were directly connected.

5.6.4 Calibration/Validation Results

Figures 5.7 and 5.8 summarize the results of the calibration/validation process for the Strathearne/Vansitmart and Fennel/East 25th test areas. Overall, a good agreement between simulated and observed response was obtained. Table 5.5 summarizes the calibrated hydrologic parameters. When extrapolated over the entire model, these parameters produced the responses illustrated in Figures 5.9 and 5.10 at the Queen/Barton and WPCP sites. Good agreement between the observed and simulated responses was also obtained at these sites. Typical simulated versus observed hydrographs for each of these four sites are included in Appendix C.

For the Plymouth Street outfall, model performance was checked qualitatively by noting whether or not CSO's occurred for the calibration/validation events. This assessment yielded 80 + percent agreement between simulated and observed responses. A quantitative assessment of peak flows and volumes could not be made at this site due to questionable flow measurements caused by harbour backwater and the opening/closing of a tide gate upstream of the site.

A final check on model performance was made by comparing simulated and observed stormwater inflows to WPCP processes for the May 1 to October 31, 1989 period. This analysis indicated simulated and observed WPCP inflow volumes of 3.5 and 3.8 million cubic metres, respectively. Therefore, the combined sewer system model realistically simulates the runoff response of the Region's combined sewer system.

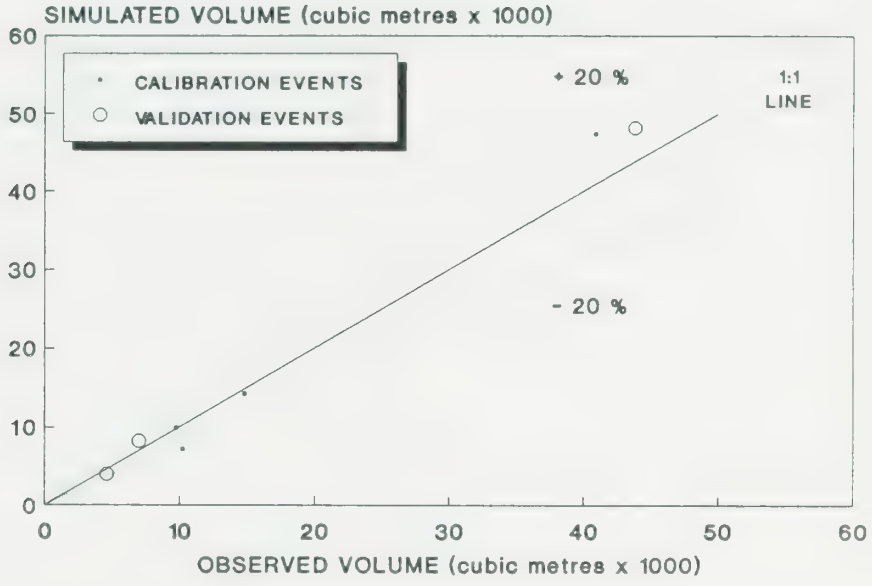
5.6.5 Time Step Selection

The time step used to model the response of a combined sewer system may greatly influence the resulting annual CSO volumes and frequencies. For example, while a short time step of 10 minutes may indicate CSO's due to 20 minutes of intense rainfall, a 60 minute time step may not. Generally, as the modelling time step increases, the trends outlined below are noted in rainfall patterns and runoff characteristics.

1. Rainfall intensities decrease because rainfall volumes are distributed over longer durations of time.

2. Peak sewer system flows decrease since runoff volumes enter the sewers over longer time periods.

FIGURE 5.7: CALIBRATION / VALIDATION RESULTS
STRATHEARNE AVE COMBINED SEWER AT VANSITMART



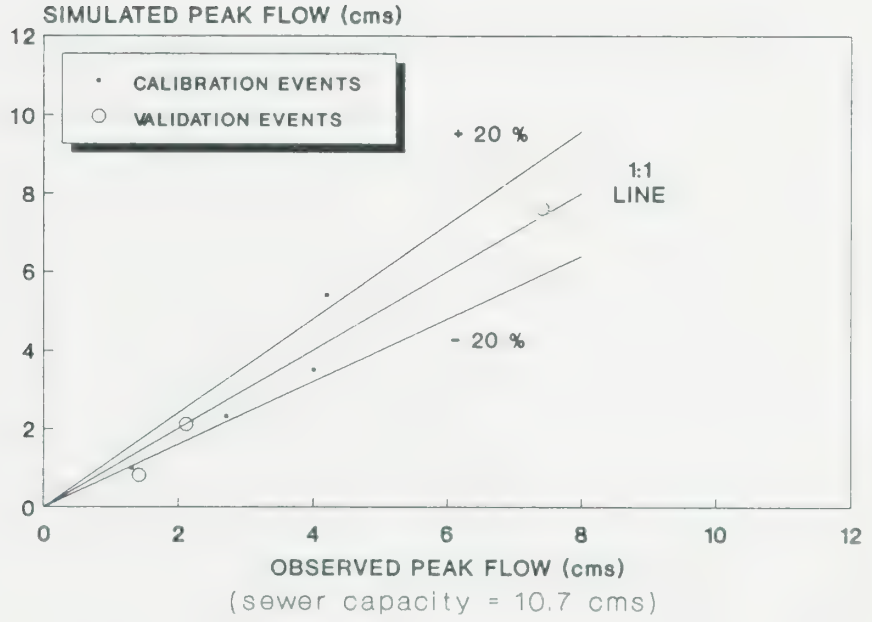
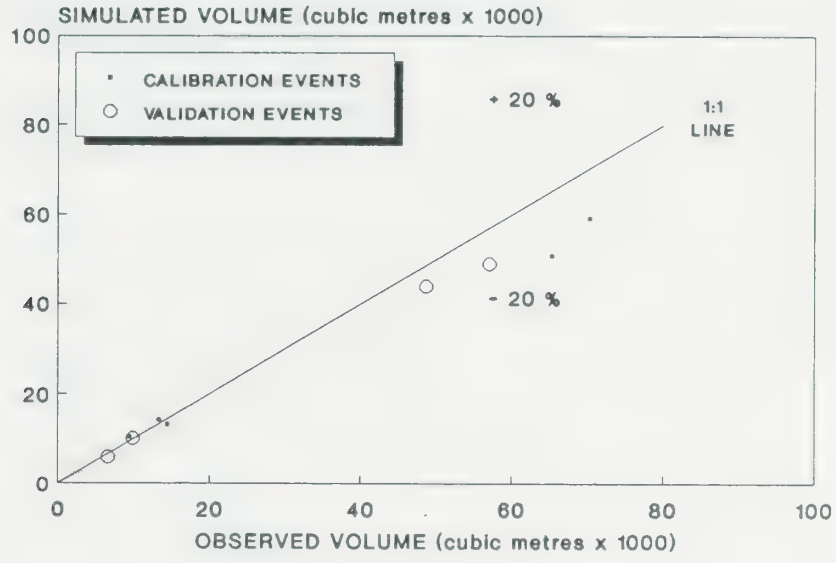


FIGURE 5.8: CALIBRATION / VALIDATION RESULTS
FENNELL AVE COMBINED SEWER AT EAST 25th



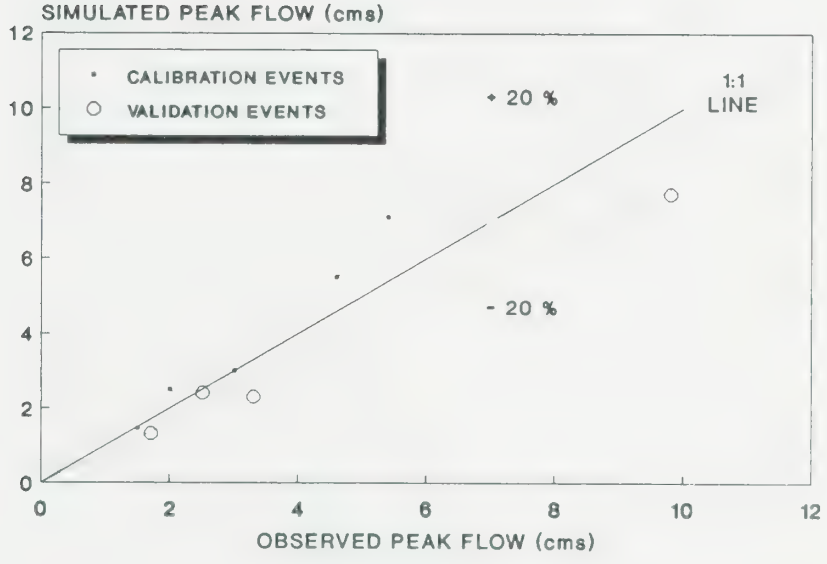
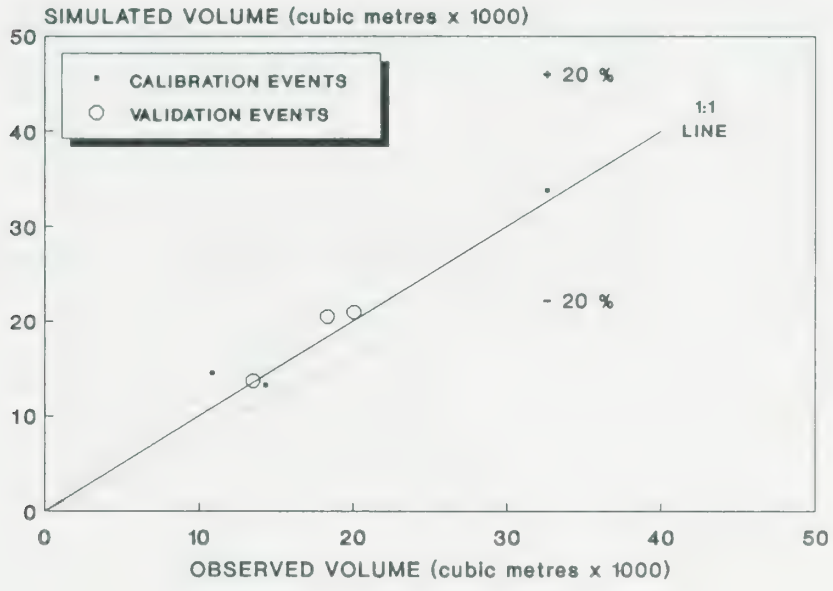


TABLE 5.5

CALIBRATED SWMM4 PARAMETERS

Item	Value
Percent Impervious for Various Land Uses	
– West Hamilton and CBD Residential Areas	52
– Mountain Residential Areas	42
– Multiple Family	70
– Industrial	80
– Commercial	85
– Parkland/Open Space	0
Percent of Impervious Area With No Depression Storage	33
Depression Storage (mm)	
– Impervious Areas	1.5
– Pervious Areas	4.0
Manning's 'n' For Overland Flow	
– Impervious Areas	0.017
– Pervious Areas	0.250
Horton Infiltration Parameters	
– Maximum Infiltration (mm/hr)	75
– Minimum Infiltration (mm/hr)	6

FIGURE 5.9: CALIBRATION / VALIDATION RESULTS WESTERN INTERCEPTOR AT QUEEN & BARTON



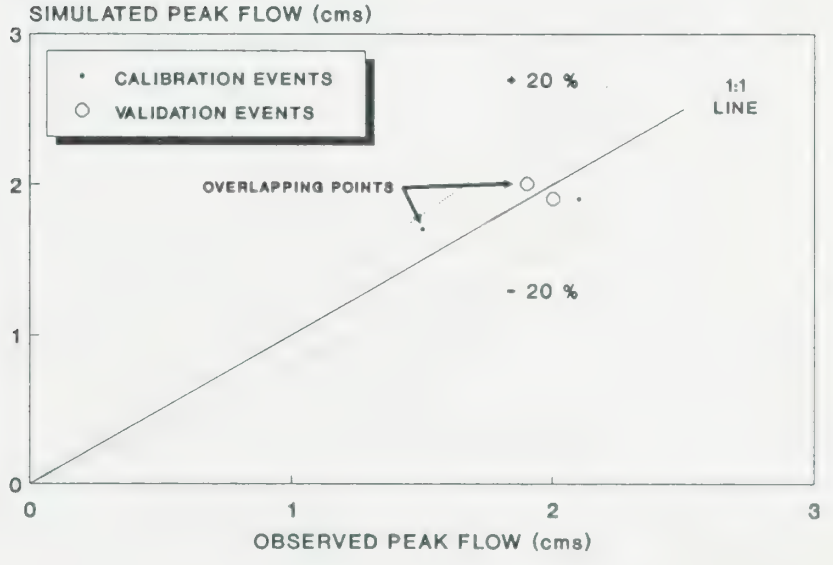
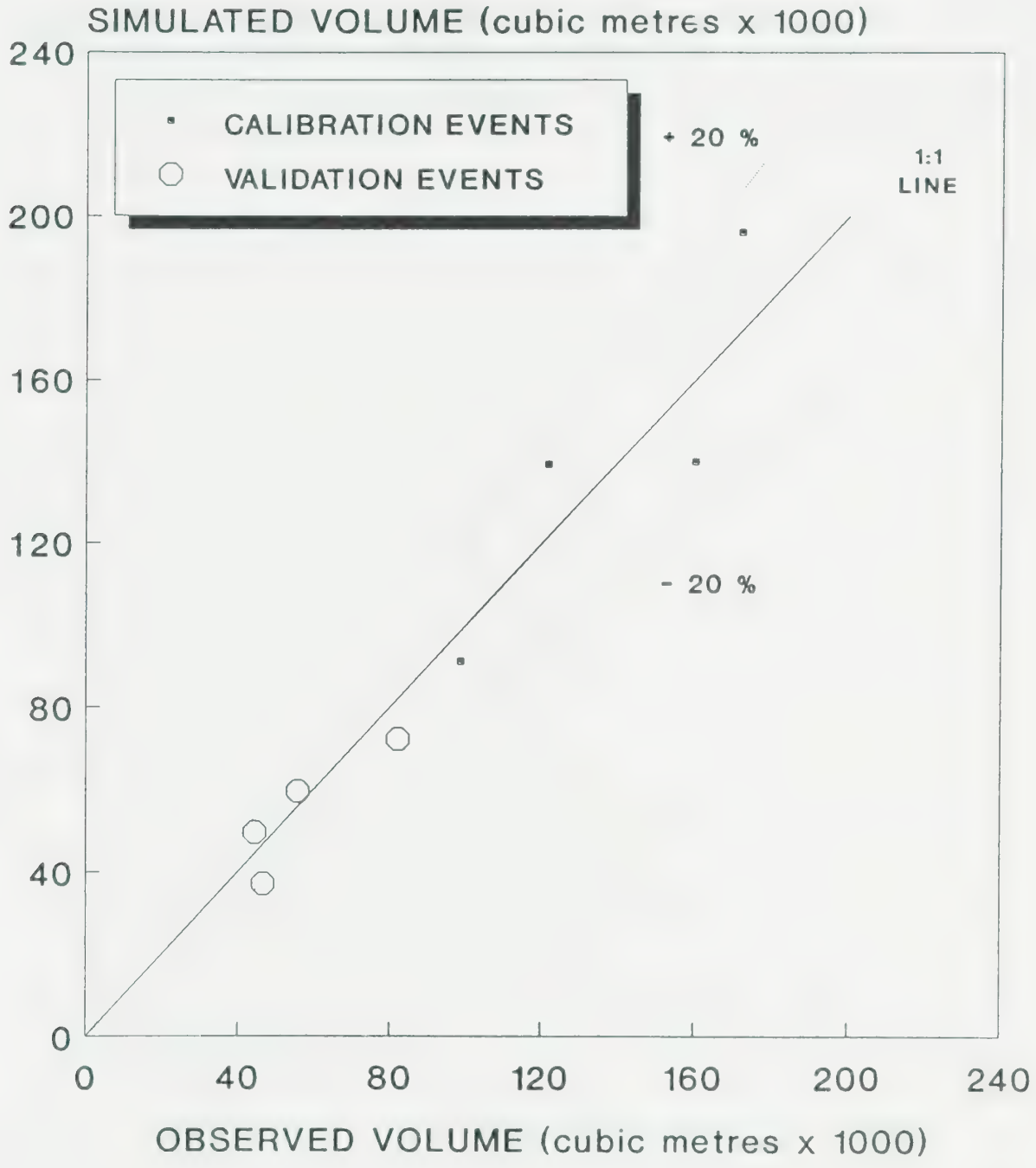


FIGURE 5.10: CALIBRATION/VALIDATION RESULTS FOR WOODWARD AVE WPCP INFLOW VOLUMES



3. Regulator threshold capacities are less likely to be exceeded as time step increases because of decreases in the modelled peak flows.

Model calibration/validation was conducted using a 10 minute time step. A 10 minute time step permitted accurate calibration of hydrologic and hydraulic parameters without the volumes of data associated with a 5 minute time step. However, time steps of 15, 20, 30 or 60 minutes may produce nearly identical results (i.e. CSO volumes and frequencies) in less time. An analysis was, therefore, conducted to determine the optimum time step for long term simulations.

The time step sensitivity analysis consisted of recomputing (using 15, 20, 30 and 60 minute time steps) Strathearne Avenue peak flows and CSO volumes for the events of July 20, 26 and 27, 1989 and comparing the results to those obtained using a 10 minute time step.

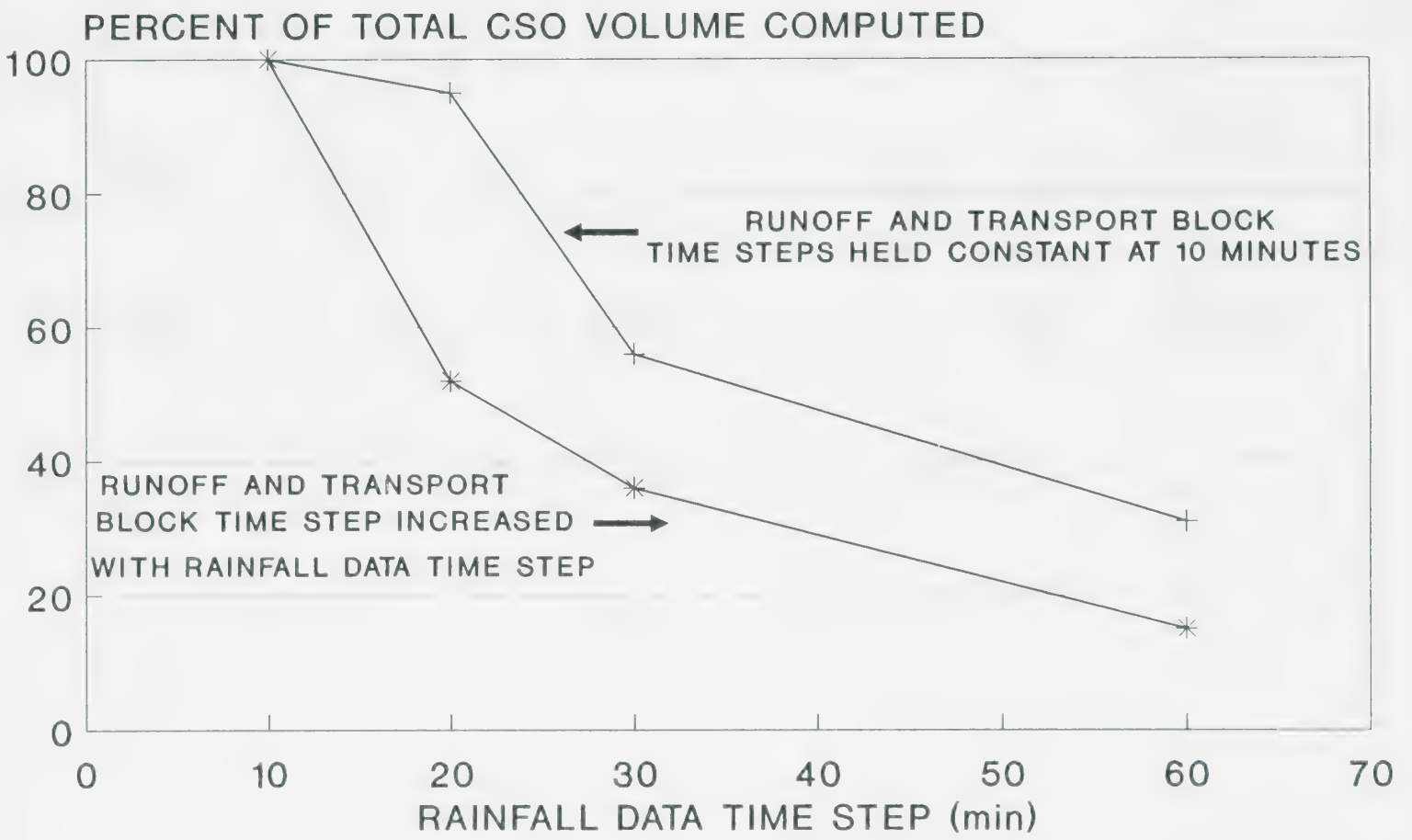
Figure 5.11 summarizes the results of the time step sensitivity analysis. Generally, as the time step increases, the probability of accurately modelling the occurrence of a CSO event, and the corresponding CSO volume, decreases. The study team will, therefore, analyze the combined sewer system using both a short time step of 10 minutes and rainfall data for the dry, typical and wet years identified earlier. This decision required that the study team to obtain the tipping bucket rain gauge charts for the years under analysis and to work the data up into three 10 minute time series. This approach to CSO analysis is different than those used in the past which normally rely on either:

- design rainfall events; or
- long term (i.e. 10 or 20 years) continuous simulations with a 1-hour time step.

However, the study team believes that it is the most appropriate, and practical, option since:

- the adverse rainfall/runoff response characteristics outlined above are avoided; and
- CSO characteristics and remedial works may be accurately assessed for a wide range of rainfall conditions.

FIGURE 5.11: TIME STEP SENSITIVITY ANALYSIS



5.7 WPCP Dynamic Model Development and Calibration Studies

5.7.1 General

The GPS Dynamic modelling was used to facilitate analysis of existing data through calibration of a detailed mathematical model. After the calibration/validation studies, the model then was used to provide performance estimates of operating conditions for different designs and operating conditions in the real plant.

5.7.2 Structure of Dynamic Mathematical Model

The model was written in ACSL (Mitchell and Gauthier Associates, 1986) based on a library of dynamic mathematical models developed at McMaster University (Patry and Takacs, 1990). The assumptions and simplifications for modelling the plant's complex layout are discussed, and the calibration – verification procedure is presented. Full documentation of the model and its calibration has been written as a separate report (Takacs et al., 1990), since the ASCL code is written in specially licensed software.

The IAWPRC and modified Vitasovic models, respectively, were used for the biological and settling processes on the plant. The IAWPRC model (Henze et al., 1987) describes the carbonaceous interactions and ammonium oxidation for biological reactors as well as other side reactions (hydrolysis, cell decay, etc.). It is based on extensive experimental research and has been tested, in a number of case studies.

The modified Vitasovic settler model (Takacs et al., 1990) was used for clarifiers. This is a dynamic settler model which was successfully calibrated previously in a number of cases to predict the underflow concentration, the sludge blanket level height, and the effluent suspended solids concentration. This settler model could simulate the behaviour of the south side (horizontal flow) settlers, which were also less sensitive to the hydraulic shock load due to storms. However, the north side consisting of 8 radial flow rectangular settlers showed a different behaviour which required modification to the settler model.

Two modifications were examined for the constant feed layer theory and modelling the "active", hydraulically overloaded portion of the settler separately (see BEAK, 1991).

5.7.3 Calibration and Verification of The Model Using SIMUSOLV

The calibration of the dynamic model was assisted by a powerful non–linear optimization package, SIMUSOLV (Steiner et al., 1987). SIMUSOLV runs the program, calculates a likelihood function based on the observed data and simulation results, then keeps re;unning the simulation modifying the parameters and trying to maximize the likelihood function. This process is extremely demanding of CPU time. Consequently, the calibration turned out to be the most time consuming of all activities in this study.

The model was calibrated using annual, monthly and event data. Calibration studies for the event data were based upon selective parameter values which minimized the residual sum of squares between observed concentrations and calculated concentrations. The test of the adequacy of the parameter values in the validation studies was based upon the calculated concentrations qualitatively agreeing with the observed concentrations for other events. This test of the adequacy of the model calibration was selected because some additional data is required to completely test the model.

Steady–State Calibration

Calibration studies were undertaken to identify a parameter set for the yearly averages. These values were checked for each month of the nine month period January – September 1989. One aspect in particular, should be underlined at this point: the real "dynamic" variables were still uncalibrated, but they have very little effect on the steady–state runs.

The results of the ten steady–state runs (nine months and yearly average) are plotted elsewhere (Beak, 1991 Takacs et al., 1990) for parameters such as primary effluent BOD, and plant effluent BOD, ammonia, nitrate and suspended solids. Representative results are given for primary and final BOD in Figure 5.12 and 5.13. The graphs show a very good match for the yearly average data points (being the calibration) and the individual months. Some modifications should be sought for some parameters when additional data are obtained in the future. These refinements are documented in Beak (1991), and Takacs, et. al., (1991).

Dynamic Calibration

From the five dynamic events the one with the highest peak flow rate (Hurricane Hugo) was selected as the main data base for dynamic calibration. It was assumed that the dynamics of the system was

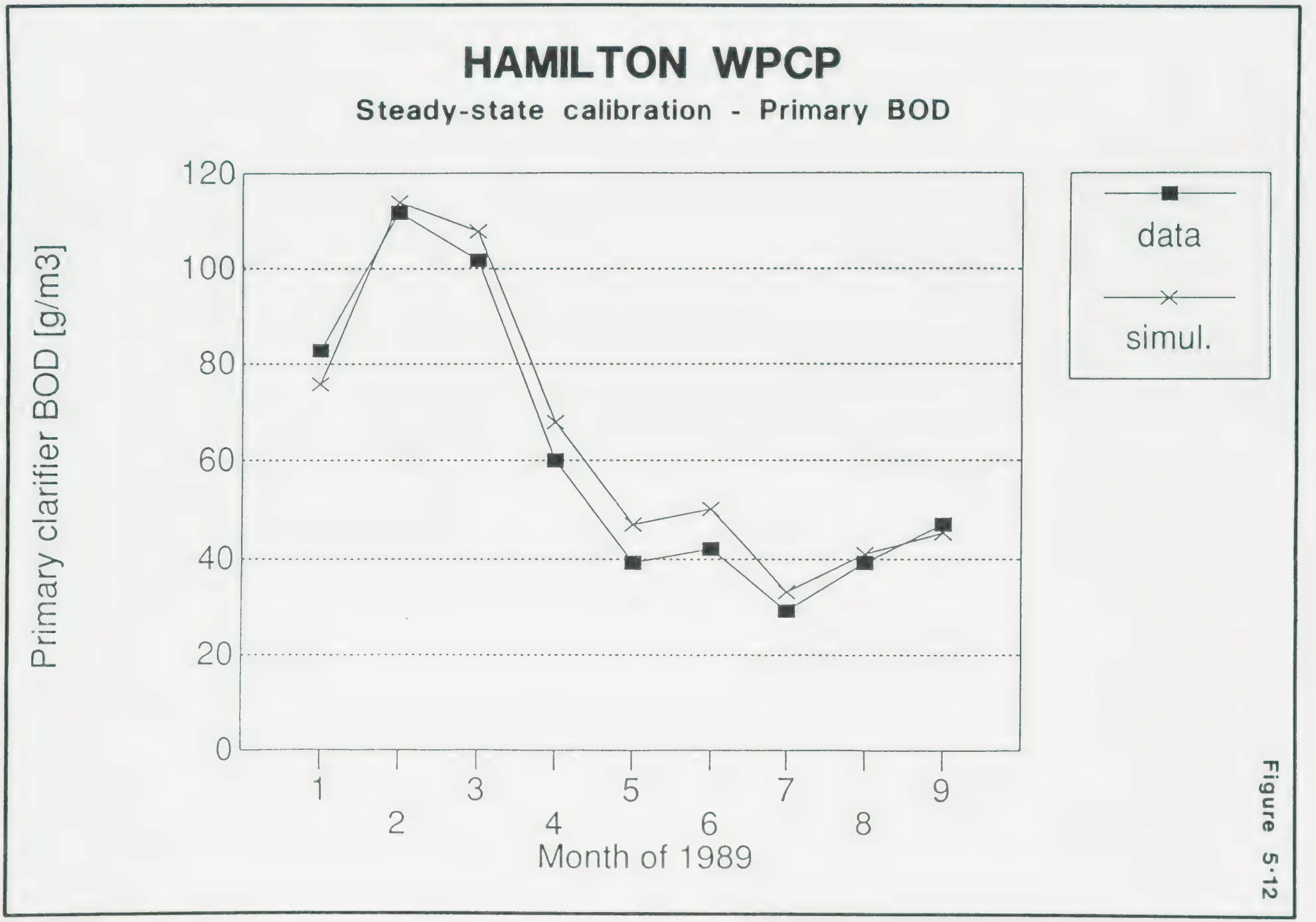

HAMILTON WPCP
Steady-state calibration - Primary BOD
Primary clarifier BOD [g/m3]
120
100
80
60
40
20
0
1
2
3
4
5
6
7
8
9
Month of 1989
data
simul.

Figure 5·12

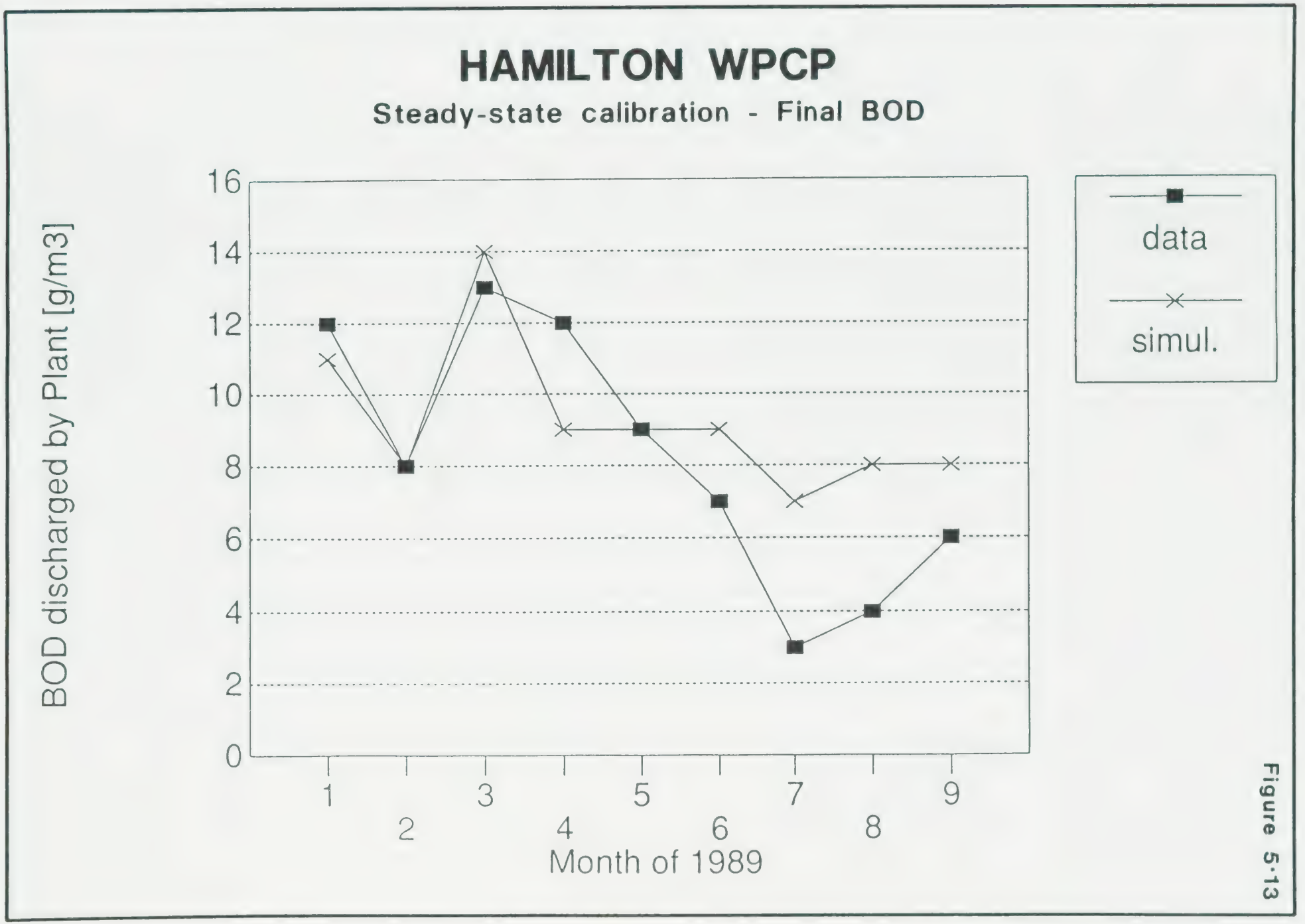

HAMILTON WPCP
Steady-state calibration - Final BOD
BOD discharged by Plant [g/m3]
16
14
12
10
8
6
4
2
0
1
2
3
4
5
6
7
8
9
Month of 1989
data
simul.

Figure 5·13

most pronounced under the heavy loading conditions encountered during this storm. Influent data during the storm are given in Beak (1991).

Dynamic calibration was done using a nonlinear optimization package called SIMUSOLV (Steiner et al., 1987).

The results of the calibration process are given in Figure 5.14 for the primary effluent while the results are given in Figure 5.15 for the same parameters (SS, BOD, ammonia, nitrate) for the combined secondary effluent from the north side of the plant. The model coefficient values and their biochemical values are defined elsewhere (Takacs et al., 1990). Dots represent actual data points, while the continuous line is the result of the calibrated simulation. Overall the agreement is good and hence the calibration is concluded to be successful. Additional calibration studies defined elsewhere could be carried out to improve the understanding of process dynamics (e.g., additional primary effluent data; and variations in the biodegradable fraction of the influent during storms).

Dynamic Verification Studies

Three more storm events (Storm # 1, 2, 3) were simulated with the calibrated parameter values derived from the Hugo storm calibration (see Beak, 1991; Takacs, et al., 1990 for their documentation). Other verification studies carried out included laboratory measurement of the secondary settling velocity (see Takacs et al., 1990).

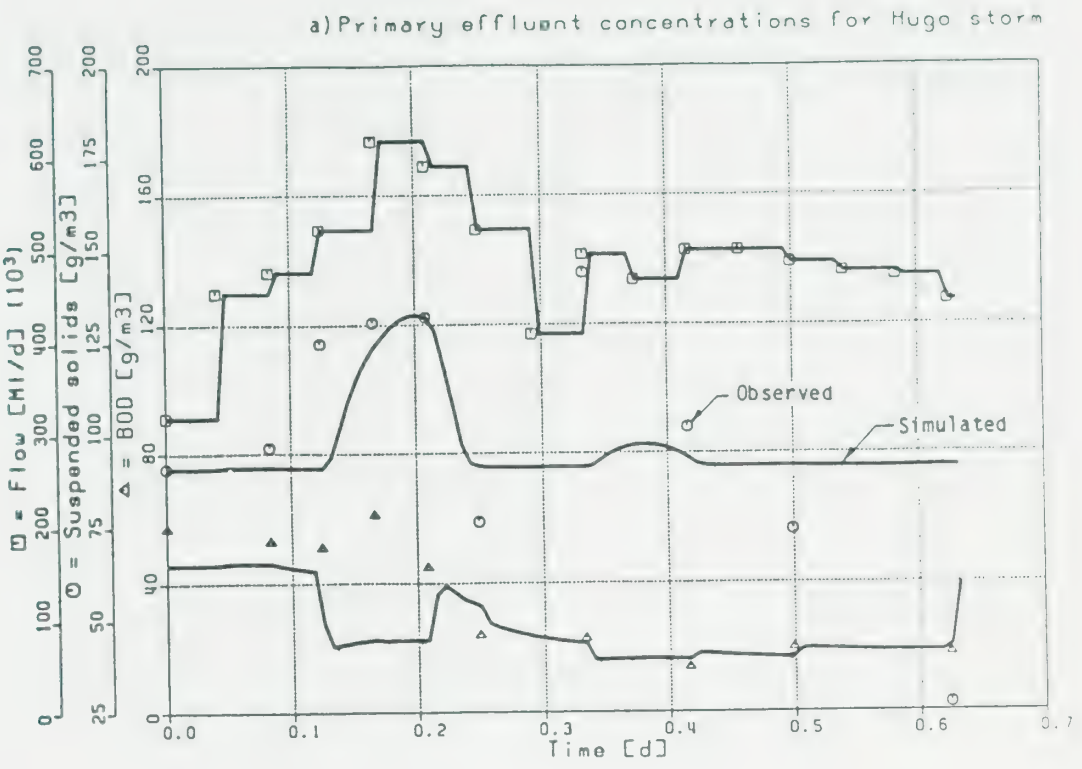
Overall the agreement was reasonable, providing confidence in use of the model for evaluating the impacts of storms upon plant performance and for sizing studies.

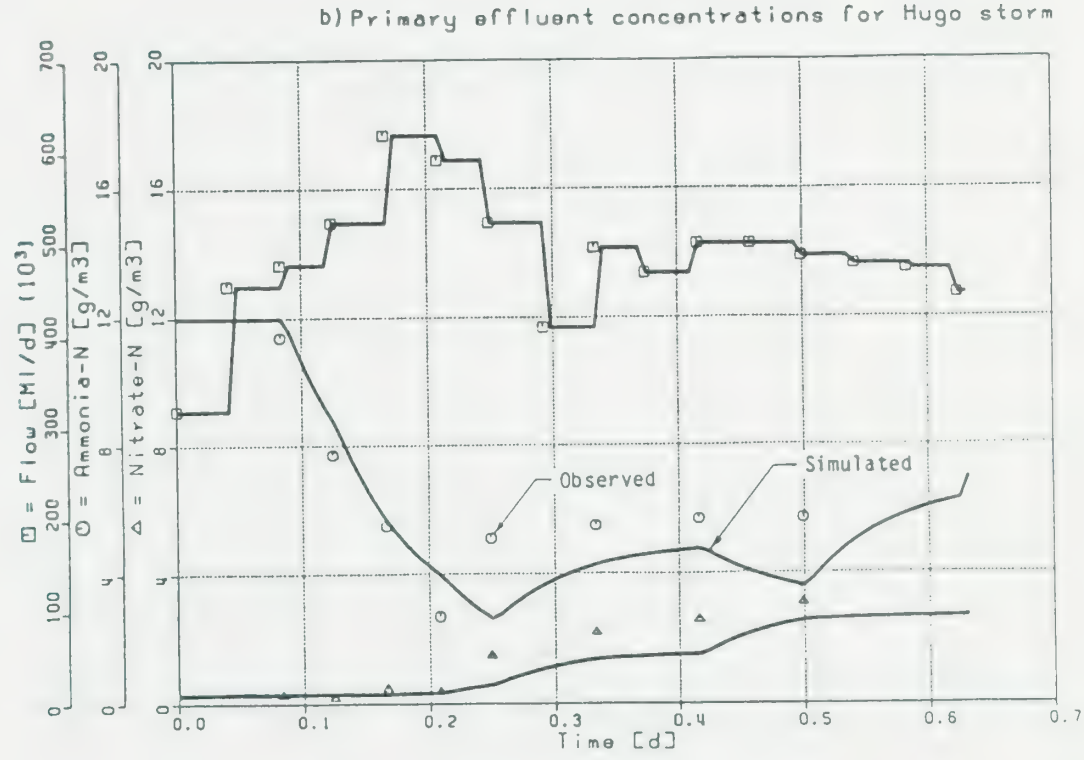
5.8 Calibration and Validation of Stormwater Spreadsheet Model

The stormwater spreadsheet model could not be calibrated directly since the entire drainage area of each basin was not included in the analysis. Rather, only those areas that are currently developed or slated for development were modelled. Therefore, flow monitoring data on local watercourses (i.e. Redhill Creek) in the area could not be used directly, as they gauge flows from all upstream areas. However, the runoff coefficients used in the model were calibrated indirectly applying:

- urban runoff coefficients based on site knowledge and calibrated parameters from both the combined sewer system model and from other studies; and
- rural runoff coefficients based partially on observed runoff responses in the area, and partly from calibrated values obtained in other studies for similar soil types.

Figure 5·14

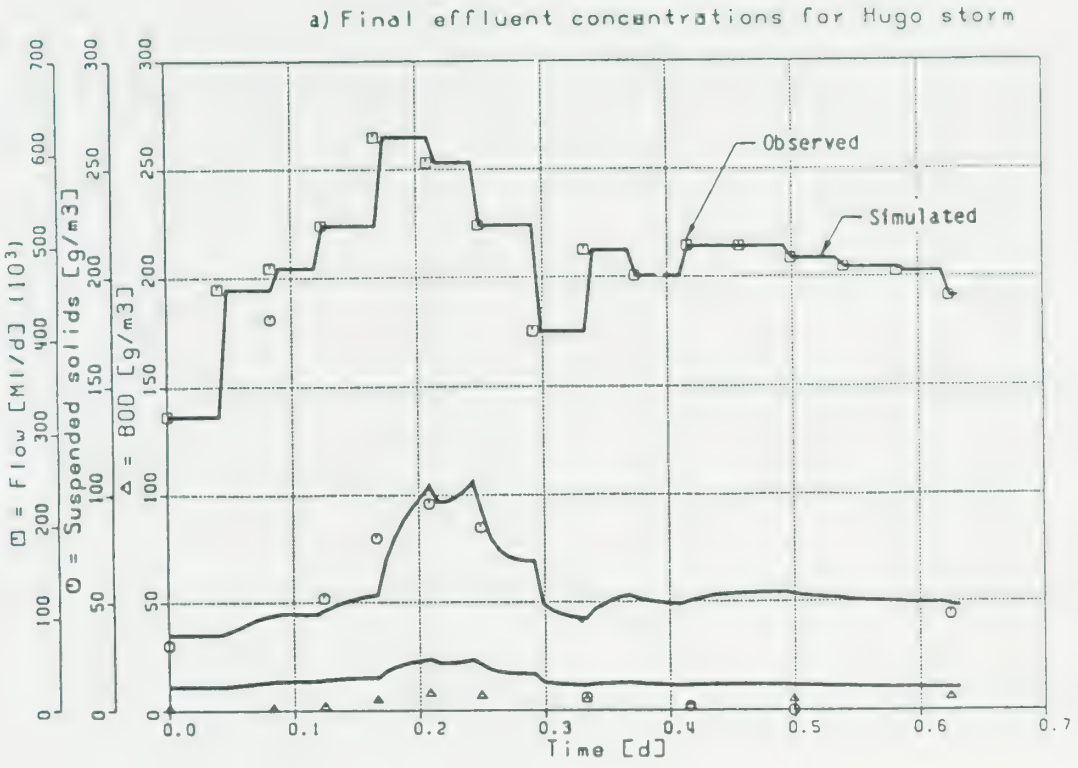


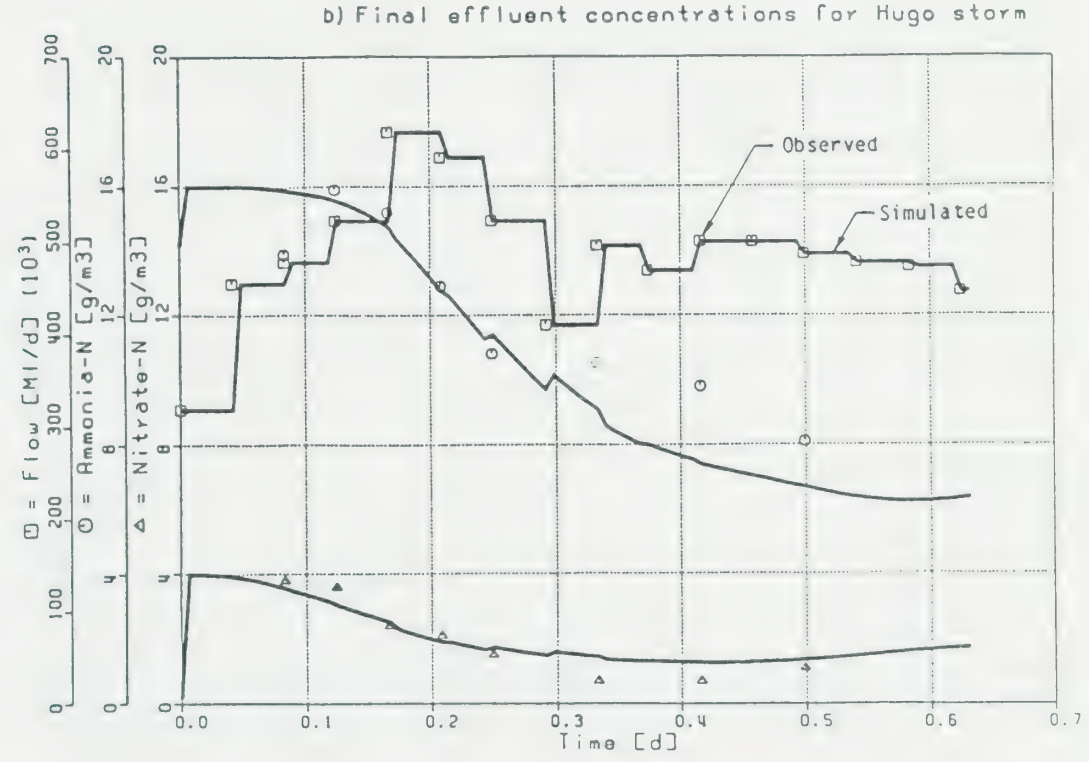


HAMILTON WPCP

Dynamic Calibration - Primary Effluent

Figure 5·15





HAMILTON WPCP

Dynamic Calibration - Final Effluent

6 – CSO, WPCP AND STORMWATER RUNOFF ANALYSIS

6.1 Combined Sewer System Analysis

6.1.1 Introduction

The next phase of the CSO analysis was to use the calibrated/validated combined sewershed model to analyze the existing system under design event and continuous situations. The objectives of the analysis are to ascertain:

1. CSO volumes and constituent loads to the receiving water bodies for various design events and rainfall patterns;
2. The distribution of CSO volumes between the various outfalls;
3. The frequency of CSO's at the various outfalls;
4. Hydraulic loads on the WPCP; and
5. The sanitary/stormwater contributions to CSO volumes.

By meeting these objectives a clear understanding of the characteristics of CSO's within the Region can be obtained. This information can subsequently be used to propose appropriate remedial works to control CSO's (see Section 9).

6.1.2 Design Event Analysis

Design event analysis of the combined sewer system was conducted for the events listed below.

1. Two–hour 5 mm and 10 mm storm events were analyzed to ascertain system response to typical rainfall events. The 5 mm and 10 mm design storms are estimated to have return periods of approximately 1–2 weeks and 2–3 weeks, respectively.

2. A two–hour 25 mm design storm was analyzed to ascertain system response to the proposed 4 CSO/year control strategy. A 25 mm rainfall event occurs within the Region every 2–3 months.

3. To determine system response to a severe rainfall event a two hour 35 mm (i.e. the proposed 1 CSO/year design storm) storm was analyzed. On average, an event of this volume is estimated to occur once during the May 1 to October 31 time period.

Assumptions During Design Event Analysis

Before design event analysis could commence assumptions regarding the operation of the WPCP–controlled sluice gates and the time period of analysis had to be made. These assumptions are reviewed below.

Since one of the objectives of the design event analysis is to determine the relative impact of the various design storms on the system, it was important to keep uniform conditions throughout the sewershed for each design event. For this reason, the study team decided to model the WPCP controlled sluice gates in the open (e.g. normal) position. This assumption allows the maximum inflow to the WPCP to be modelled.

Design event analysis of the system also requires that some time period be selected in order to ascertain overall inflows to both the system and the WPCP. Based on the data presented in Section 4 the 12–hour period following the start of a design storm event was selected as the time period for analysis.

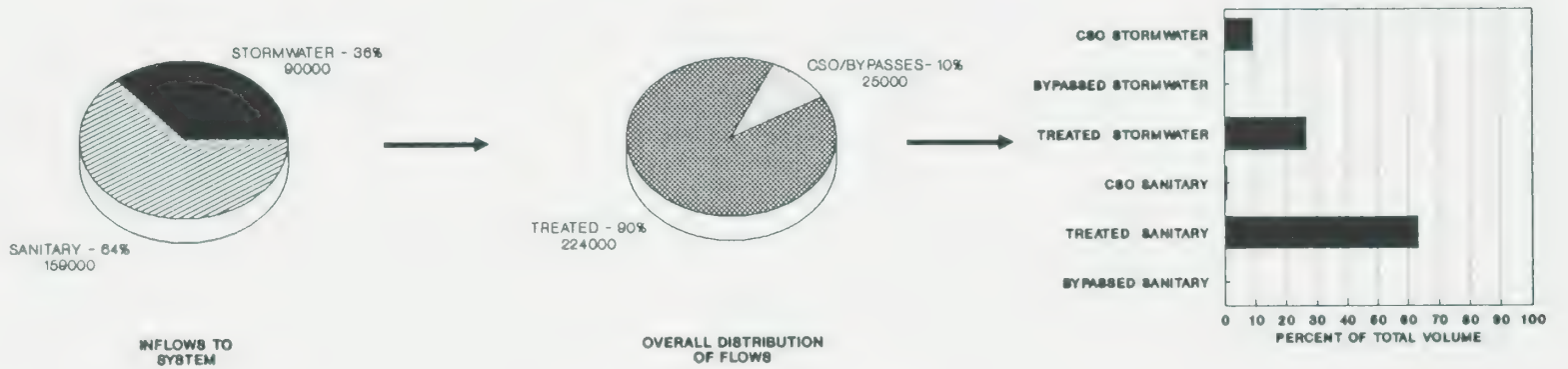
System Response to Design Events

Figures 6.1 and 6.2 summarize the overall response of the combined sewer system to the design events. General conclusions which may be drawn from these figures are outlined below.

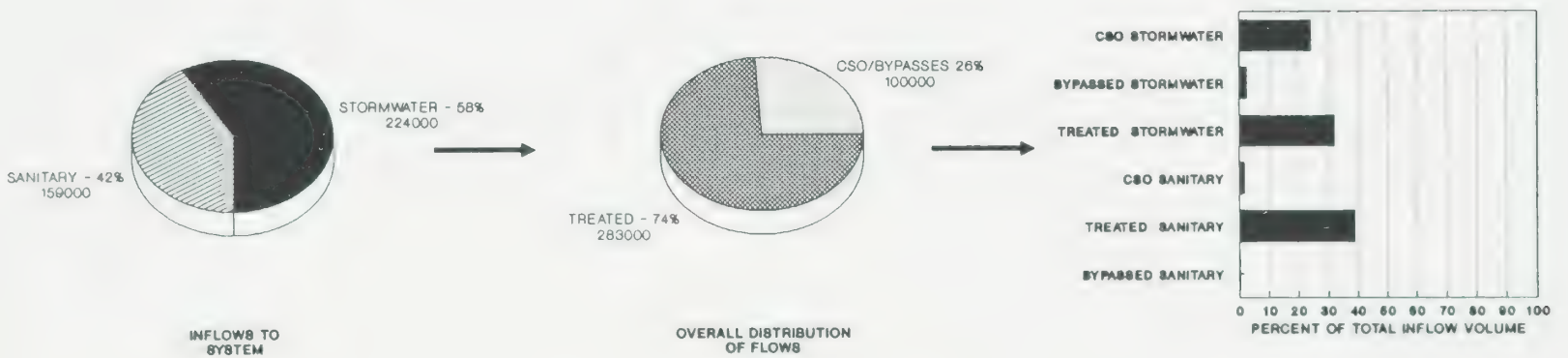
1. As the rainfall depth increases from 5 to 35 mm, the contribution of sanitary flows to the (12–hour) inflow to the system decreases from 64 percent to 15 percent. This observation indicates sanitary flow is a major/minor component of system flows during small/large rainfall events.

2. The existing combined sewer system has sufficient capacity to convey and treat in excess of 90 percent of all inflows to the system for a 5 mm event. Conversely, two thirds of all inflows during a 35 mm event will be routed to the receiving water bodies with little or no treatment.

FIGURE 6.1: DISTRIBUTION OF SANITARY AND STORMWATER FLOWS FOR 5 AND 10 mm EVENTS



a) 5mm RAINFALL EVENT



b) 10 mm RAINFALL EVENT

NOTE: VALUES UNDER FLOW LABELS ARE VOLUMES (CUBIC METRES)

3. The bypassing of combined sewage around the WPCP treatment processes increase from approximately 2 to 12 percent of all inflows for the 5 and 35 mm design events respectively. This observation illustrates the potential importance of WPCP bypass treatment systems (i.e. chlorination) to treat excess WPCP inflows. More discussions on this topic is presented in the continuous modelling section.

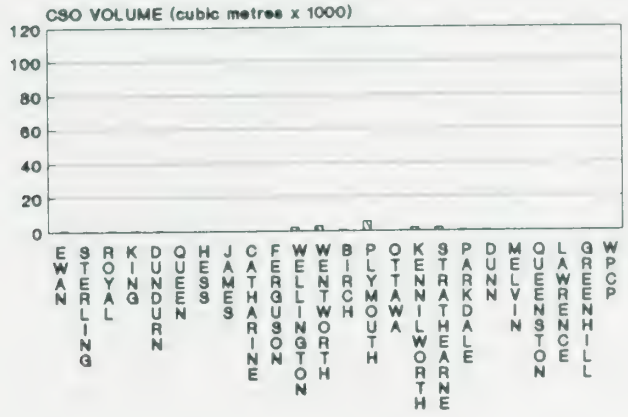
Distribution of CSO Volumes

Figure 6.3 illustrates the distribution of CSO volumes for the 5, 25 and 35 mm design events. The CSO volumes generally represent the volume of combined sewage which would be discharged to the receiving waters untreated. For the Greenhill storage facility/CSO outfall, the illustrated CSO volume represents the combined sewage volume which is discharged to Redhill Creek after the 80,000 m^3 facility is full. CSO flows exiting the Greenhill tank are also partially treated by the primary treatment capabilities of the facility. For the WPCP, the CSO volumes represent those volumes which are expected to by bypassed around the plant's treatment processes (i.e. pumphouse/plant bypasses). These flows are expected to receive chlorination prior to being discharged to Hamilton Harbour. Generally, the greatest CSO volumes are associated with large and impervious sewersheds (e.g. Wentworth) while much lower CSO volumes are associated with the smaller and more pervious sewersheds (e.g. Ewan). Figure 6.3 also indicates whether or not the outfalls are located near human contact areas or industrial/other areas. More information on the facilities by each "human contact" outfall is included later in this section. The total CSO volumes included in Figure 6.3 illustrate the large volumes which will ultimately require treatment. Sections 8 and 9 addresses the required WPCP modifications to treat these volumes.

CSO Sewage Strength

A planning level analysis of the design events was also conducted to estimate the contributions of sanitary and stormwater flows to CSO's and WPCP bypasses. The results listed in Table 6.1 suggest that sanitary sewage will generally comprise approximately 20 to 5 percent of the CSO volume for frequent and infrequent rainfall events, respectively. The sanitary/stormwater ratio is important as it is required to determine the EMC of CSO's using the selected EMC's for sanitary and stormwater flows. As detailed in Table 6.1, CSO sewage has much lower constituent concentrations than sanitary sewage, and constituent concentrations in CSO's generally decrease as rainfall volumes increase. Appendix I provides additional insight with respect to the strength of CSO sewage which may be expected within the Region.

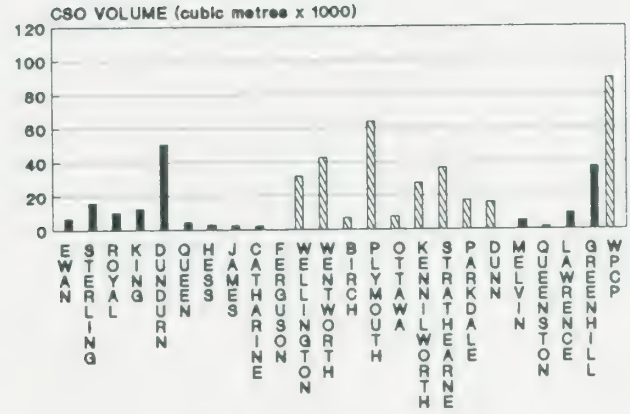
FIGURE 6.3: DISTRIBUTION OF CSO VOLUMES AT OUTFALLS FOR 5, 25 AND 35 MM EVENTS



5 mm EVENT

TOTAL CSO VOLUME 25,000 CUBIC METRES

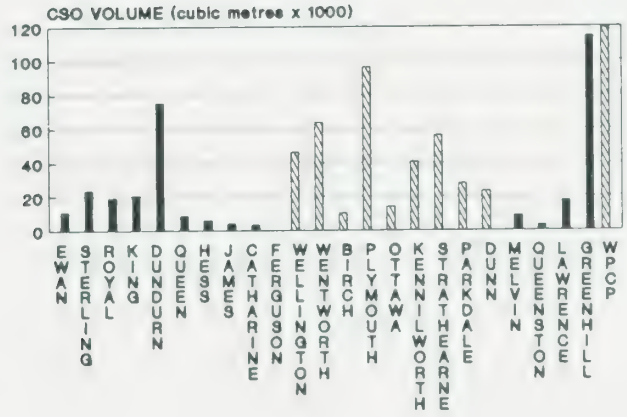
OUTFALLS BY INDUSTRIAL & OTHER AREAS



25 mm EVENT

TOTAL CSO VOLUME 499,000 CUBIC METRES

OUTFALLS BY HUMAN CONTACT AREAS



35 mm EVENT

TOTAL CSO VOLUME 812,000 CUBIC METRES

TABLE 6.1

CSO CHARACTERISTICS

Rainfall Depth (mm)	Contribution of Specified Flow to CSO and WPCP Bypass Volumes		Estimated Concentration of Stated Constituent in CSO's and WPCP Bypasses (mg/l)		
	Sanitary (%)	Stormwater (%)	SS	TP	TKN
5	17	83	134	1.29	5.8
10	9	91	118	0.84	3.7
25	8	92	115	0.78	3.4
35	7	93	114	0.72	3.2
Strength of Raw Sanitary Sewage			302	6.0	27.4

Note: See Appendix I for additional information on CSO characterisitcs.

6.1.3 Continuous Modelling Analysis

Continuous modelling of the combined sewer system was conducted for the dry, typical and a wet years introduced in Section 5. The selected years have rainfall depths of 290, 454 and 706 mm, respectively. The WPCP controlled sluice gates were assumed to be in the open (i.e. normal) position during all analysis.

Overall System Response

Figure 6.4 summarizes the overall response of the combined sewer system to a dry, wet and typical year. General conclusions which may be drawn from the figure are outlined below.

1. Sanitary sewage is the dominant source of inflow to the system. This trend may be explained by considering two factors. First, sanitary flows are continuous, while stormwater flows are intermittent. Therefore, while stormwater volumes may exceed sanitary sewage volumes over the short term (i.e. during rainfall events), they do not exceed sanitary volumes over the long term. Secondly, the Woodward Avenue WPCP receives sanitary flows from an area approximately 50 percent larger than the combined sewershed draining to the WPCP, as it includes such separated sewer areas as Ancaster and Stoney Creek.

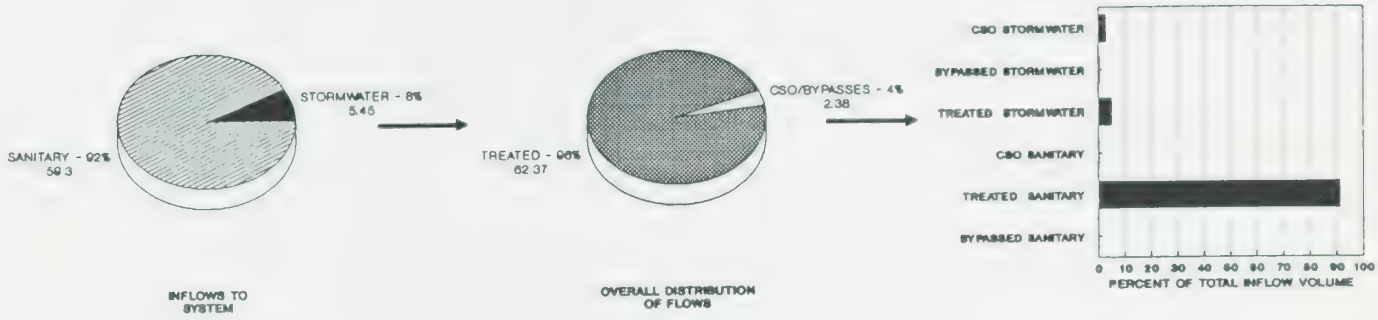
2. While annual CSO and WPCP bypass volumes are large and vary from approximately 2 to 8 million cubic metres, they appear small when compared to the volumes of sewage treated at the WPCP.

To conclude, Figure 6.4 illustrates the magnitude of flows which pass through the WPCP before being discharged to Hamilton Harbour. This observation suggest constituent loadings to the receiving water bodies due to WPCP effluent may be significantly larger than loading from either CSO's or stormwater discharges.

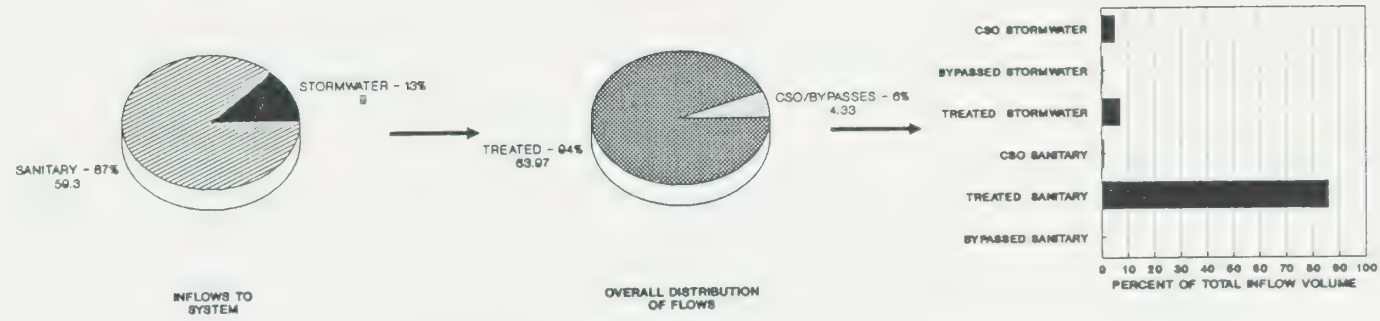
Frequency of CSO's

The frequency of CSO's was ascertained at each outfall and is illustrated in Figure 6.5. Generally, the outfalls with the highest number of CSO events are downstream of flow regulators with the lowest threshold capacities. In all cases, each outfall, including the WPCP and the Greenhill Avenue tank, currently discharge CSO's to the receiving water bodies more frequently than the general guidelines

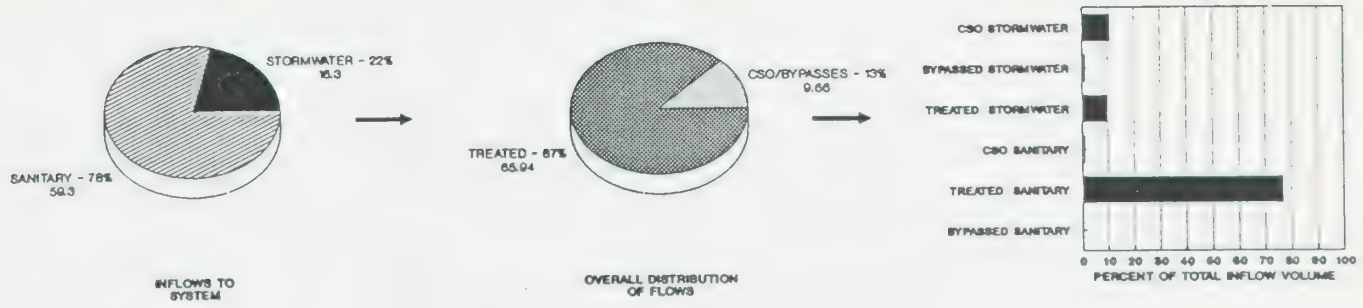
FIGURE 6.4: DISTRIBUTION OF SANITARY AND STORMWATER FLOWS FOR DRY, TYPICAL AND WET YEARS



a) DRY YEAR - 1971 - 294 mm RAINFALL



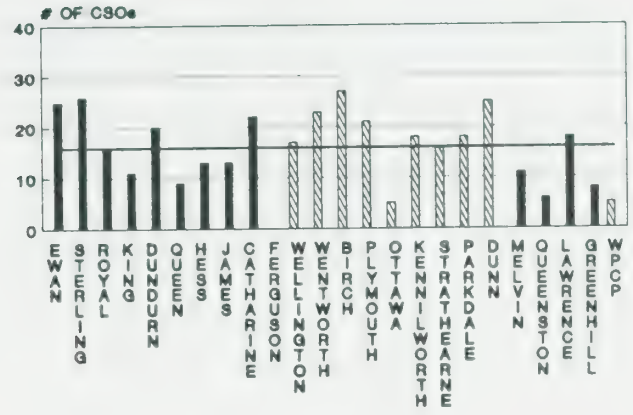
b) TYPICAL YEAR - 1989 - 454 mm RAINFALL



c) WET YEAR - 1981 - 706 mm RAINFALL

NOTE: VALUES UNDER FLOW LABELS ARE VOLUMES IN MILLIONS OF CUBIC METRES

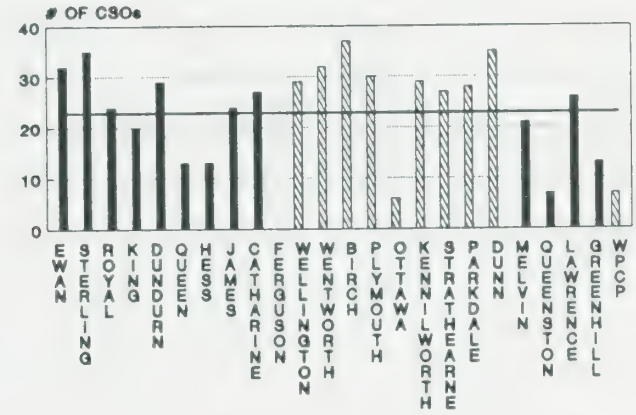
FIGURE 6.5: FREQUENCY OF COMBINED SEWER OVERFLOWS AT OUTFALLS



DRY YEAR
1971
(294 mm RAINFALL)

TOTAL # OF CSO EVENTS
384

OUTFALLS BY INDUSTRIAL & OTHER AREAS

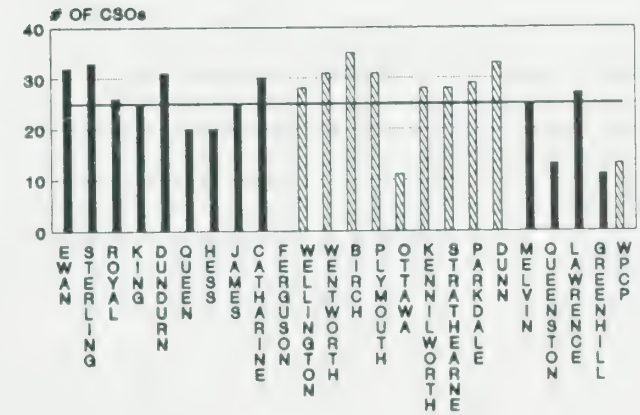


TYPICAL YEAR
1989
(454 mm RAINFALL)

TOTAL # OF CSO EVENTS
562

OUTFALLS BY HUMAN CONTACT AREAS

AVERAGE NUMBER OF CSO EVENTS



WET YEAR
1981
(706 mm RAINFALL)

TOTAL # OF CSO EVENTS
610

of 1 or 4 CSO events per year. This observation indicates remedial works to control the frequency (and volumes) of CSO events will be required for each outfall.

CSO's occur at each outfall, on average, 23 times during a typical year. This CSO frequency is equivalent to 1 CSO event every 8 days. The interevent time between CSO events increases to 11.5 days and decreases to 7.5 days during a dry and wet year respectively. This observation, when combined with the total CSO volumes, indicates that while fewer and smaller CSO events occur during a dry year, CSO events occur at about the same frequency during a wet year, although the CSO volumes during each event are larger.

Of special interest is the frequency of CSO's at the outfalls where current and future human contact areas exist. These outfalls, which are outlined in Table 6.2, will require highest level of CSO reduction in order to meet the proposed MOE objectives of 1 CSO/year for sensitive areas.

Distribution of CSO Volumes

The distribution of CSO volumes at the various outfalls for the three years under analysis is illustrated in Figure 6.6. The most noticeable trend from this Figure is the three and a half fold increase in CSO volumes from a dry to a wet year. This trend illustrates CSO volumes to the receiving water bodies can vary substantially from year to year and are highly sensitive to rainfall depths. Constituent loads to the receiving water bodies are presented later in this section. Also evident from Figure 6.6 is the relative contribution of each outfall to the total CSO volume. Generally, the outfalls which contribute the majority of CSO's to the receiving water bodies have the largest and most impervious drainage areas upstream. A detailed breakdown of the distribution of CSO volumes is included in Table 6.3. This table indicates approximately 15, 28, 56 and 85 percent of the annual CSO volume is discharged from the top 1, 3, 5 and 10 outfalls, respectively. Therefore, half of the CSO outfalls are minor and collectively contribute less than 15 percent of the CSO volume. A review of Figure 6.6 indicates CSO volumes near the human contact areas identified in Table 6.2 are generally smaller than in industrial areas. This observation creates a conflict between the human contact and other areas when remedial works are considered. For example, CSO remedial works are often prioritized using CSO volumes and the potential for human contact. Therefore, although remedial works at the sensitive outfalls may decrease the chance of human contact with contaminated waste, the works would do little to substantially reduce CSO volumes to the receiving water bodies. Conversely, remedial works on many of the industrial outfalls would decrease CSO volumes substantially, but would only marginally decrease the potential for human contact with CSO's.

TABLE 6.2

FREQUENCY OF CSO'S NEAR HUMAN CONTACT AREAS

Outfall Name	Type of Facilities Near Outfall	Frequency of CSO's During a Typical Year
Ewan	Spencer Creek, Town of Dundas	24
Royal	Parkland and Golf Course	
Sterling	Ravine Lands	35
King and Dundurn	Park Land	20 29
Queen and Hess	Lac Property (future marina)	13 13
James	Existing Marina	24
Catharine	Royal Canadian Navy Docks	21
Melvin, Queenston, Lawrence and Greenhill	Redhill Creek Valley	7

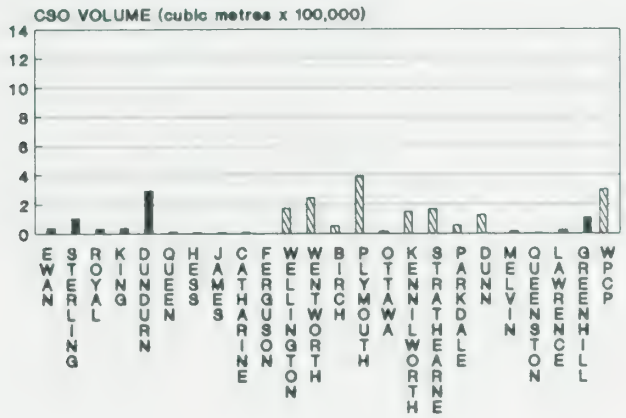
TABLE 6.4

SANITARY AND STORMWATER CONTRIBUTIONS TO CSO's

Year	Contribution of Specified Flow to CSO and STP Bypass Volumes	
	Sanitary (%)	Stormwater (%)
DRY (1971)	7	93
TYPICAL (1989)	7.5	92.5
WET (1981)	7.5	92.5

Note: See Appendix I for additional information with respect to sanitary and stormwater contributions to CSO.

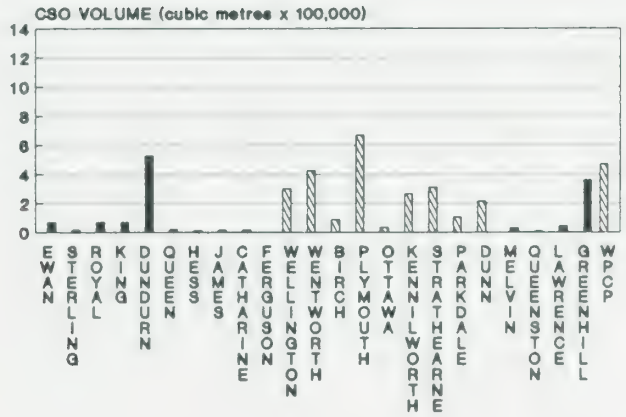
FIGURE 6.6: DISTRIBUTION OF CSO VOLUMES AT OUTFALLS FOR DRY, TYPICAL AND WET YEARS



DRY YEAR
1971
(294 mm RAINFALL)

TOTAL CSO VOLUME
2,380,000
CUBIC METRES

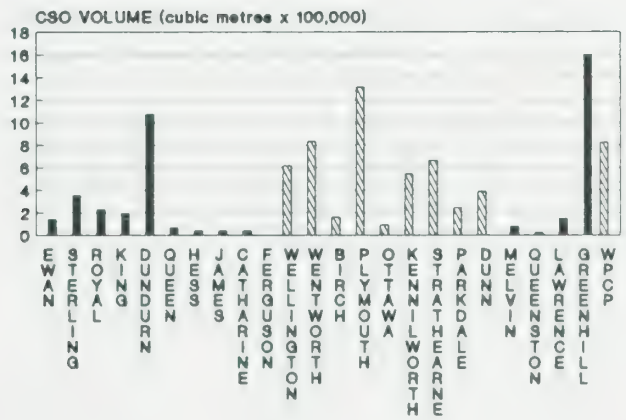
OUTFALLS BY INDUSTRIAL & OTHER AREA



TYPICAL YEAR
1989
(454 mm RAINFALL)

TOTAL CSO VOLUME
4,380,000
CUBIC METRES

OUTFALLS BY HUMAN CONTACT AREAS



WET YEAR
1981
(706 mm RAINFALL)

TOTAL CSO VOLUME
9,660,000
CUBIC METRES

TABLE 6.3

DISTRIBUTION OF CSO VOLUMES AT

OUTFALLS DURING A TYPICAL (1989) YEAR

(MAY 1 TO OCTOBER 31)

Outfall	CSO Volume (m^3)	Percent of Annual Volume	Cumulative Percent Of Annual Volume
Plymouth	669,000	15.3	15.3
Dundurn	529,000	12.1	27.4
WPCP	470,000	10.7	38.1
Wentworth	427,000	9.7	47.8
Greenhill	360,000	8.2	56.0
Strathearne	309,000	7.0	63.0
Wellington & Ferguson	301,000	6.9	69.9
Kenilworth	265,000	6.0	75.9
Dunn	213,000	4.9	80.8
Sterling	187,000	4.3	85.1
Parkdale	104,000	2.4	87.5
Birch	87,600	2.0	89.5
King	73,000	1.7	91.2
Royal	69,800	1.6	92.8
Ewan	68,700	1.6	94.4
Queenston	67,400	1.5	95.9
Lawrence	46,000	1.0	96.9
Ottawa	36,400	0.8	97.7
Melvin	31,500	0.7	98.4
Queen	23,000	0.5	98.9
James	16,500	0.4	99.3
Catharine	16,500	0.4	99.7
Hess	13,200	0.3	100.0
TOTAL	4,383,000	100.0	

WPCP plant bypassing volumes during the dry, typical and wet years are also substantial and deserve comment. First, the illustrated volumes are probably over-estimated since the process of closing the WPCP controlled sluice gates during severe rainfall events, which directs sewage away from the WPCP, was not modelled during the analysis. Consequently, the model bypassed the excess WPCP inflows around the plant. Ballpark estimates suggest "actual" plant bypass volume may be 50 percent of the computed values. Nonetheless, the analysis indicates the substantial volume of combined sewage that is directed to the receiving water bodies as a result of plant bypassing and gate closures. The magnitude of the potential plant bypassing volumes also suggests that partial treatment of these flows would help to reduce constituent loadings to the receiving water bodies.

Sanitary and Stormwater Contribution to CSO's

Table 6.4 lists the relative contributions of sanitary and stormwater flows to CSO's during a dry wet and typical year. Overall, the sanitary/stormwater contributions to CSO volumes essentially remain unchanged. Comparing the values in Table 6.4 to Table 6.1 suggests the majority of CSO volume originates during large (i.e. greater than 10 mm) rainfall events. This trend has been observed in similar studies where 40 percent of the CSO volume has been found to originate from the 6 largest events in a typical year. For this study CSO's will, be assumed to consist of 7 percent sanitary sewage and 93 percent storm water. Additional information with respect to sanitary and stormwater contributions to CSO's is contained in Appendix I.

Constituent Loadings

Constituent loadings to the receiving water bodies for the 10 water qualities constituents are detailed in Table 6.5. The loadings as extremely variable, depending on whether a dry, typical or wet year is chosen for analysis. This trend is observed because constituent loadings due to CSO's are primarily a function of CSO volumes, rather than constituent concentrations.

A comparison of the loadings made in this study for CSO's to Hamilton Harbour with those of previous RAP studies is also presented in Table 6.5. Generally both sets of estimates are within the same order of magnitude. The largest difference between the loadings estimated for a typical year, and those estimated by RAP, is for Total Phosphorus (TP).

TABLE 6.5

TOTAL CONSTITUENT LOADINGS TO RECEIVING WATER BODIES

FROM CSO'S AND WPCP BYPASSES

Constituent	Loading from May 1 to October 31 for						Estimated Loadings in RAP Documents (1987) (kg/day)
	Dry (1971) Year		Typical (1989) Year		Wet (1981) Year		
	Total (kgx1000)	Daily (kg/day)	Total (kgx1000)	Daily (kg/day)	Total (kgx1000)	Daily (kg/day)	
SS	271	1,475	500	2,715	1,100	5,985	2,200*
DS	847	4,600	1,560	8,480	3,440	18,700	17,000
TP	1.7	9.2	3.2	17	7.0	38	90*
NH_3	3.0	16	5.5	30	12.1	66	–
TKN	7.6	41	14.0	76	30.9	168	200
Zn	0.8	4.2	1.45	7.9	3.2	17.4	20
FC (total counts)	$2.2x10^{15}$	$1.2x10^{13}$	$4.0x10^{13}$	$2.2x10^{13}$	$9x10^{15}$	$4.9x10^{13}$	–
BOD_5	40.5	220	74.5	405	164	893	480
Pb	0.33	1.8	0.61	3.3	1.3	7.3	3.5
Cu	0.09	0.5	0.17	0.9	0.37	2.0	1.6

Notes: 1. Values with a * are from RAP studies, other values are from reports referenced by RAP.

2. See Appendix I for additional information regarding the range of constituent loadings to the receiving water bodies.

Note:
The constituent loadings presented in Table 6.5 are estimates based on the planning level of analysis adopted for the PCP study. The loading estimates are based on the assumptions outlined earlier and are expected to be of the right order of magnitude. However, various processes/conditions which may influence the constituent loadings were not accounted for and hence actual loadings to Hamilton Harbour may differ from those presented here. See Appendix I for additional discussion of this topic.

6.2 Possible Future Flows to be Handled by Woodward WPCP

This section presents estimates of:

1. Increases in sanitary flows to the WPCP under ultimate population conditions.

2. The magnitude (i.e. peak flow rate and volume) of the flows entering the plant for various events.

6.2.1 Population Growth Effects

Forecasted flow rates due to population growth and due to CSO diversions are also given in Table 6.6. An increase in the population served to 500,000 has been estimated for the year 2025. This is projected to increase the flow to the plant from 1989 values of approximately 315 MLD (70 MIGD) to 430 MLD (95 MIGD) (see item 1, Table 6.6).

6.2.2 Simulated Flows Reaching Plant for Various Rainfall Events

Flow patterns to the plant which are analogues of Hurricane Hugo (1.0 Times, 1.5 Times, 2.0 Times) or from two specific storms centred over the City are next summarized in Table 6.6. These five flow patterns are listed because they were used later in this study (Appendix G) to evaluate the effects upon process performance of different types of transient conditions in the plant. The "As Is" 25 mm storm is the flow pattern expected at the plant for a 25 mm storm centred over the combined sewer draining to the plant, if the overflow regulators were set as observed in the summer/fall period of 1989. The "All Flow" 25 mm storm is the flow pattern expected at the plant if all overflows from a 25 mm

TABLE 6.6

SUMMARY OF HYDRAULIC LIMITING FACTORS,

EXISTING PLANT LOADS AND PROJECTED PLANT LOADS

1. Population Growth Based Expansion Requirements

• Year 2006 (Population Forecast: 460,000) =		85 MIGD	383 MLD
• Year 2025 (Population Forecast: 500,000) =		95 MIGD	428 MLD

2. Plant Expansion Requirements for Present Storm Patterns

a) Simulated flows for typical storms

• Hurricane Hugo	Q_{avg}	99 MIGD	444 MLD
	Q_{peak}	137 MIGD	615 MLD
• 1.5 Times Hurricane Hugo	Q_{avg}	139 MIGD	627 MLD
	Q_{peak}	205 MIGD	922 MLD
• 2 Times Hurricane Hugo	Q_{avg}	180 MIGD	811 MLD
	Q_{peak}	273 MIGD	1230 MLD
• "As Is" 25 mm Storm	Q_{avg}	120 MIGD	542 MLD
	Q_{peak}	331 MIGD	1490 MLD
• "All Flow" 25 mm Storm	Q_{avg}	172 MIGD	776 MLD
	Q_{peak}	492 MIGD	2210 MLD

3. Plant Expansion Requirements for the Storage and Treatment Option (Storm Surge Directed to Plant)

a) System storage and WPCP Treatment for 4 Overflows per year (Storage = 500 ML)

• Base Flow, 2025 = 420 MLD	
• Base Flow plus treat in 1 day (420 + 500)	920 MLD
• Base Flow plus treat in 3 day (420 + 170)	590 MLD

TABLE 6.6 – CONT'D

SUMMARY OF HYDRAULIC LIMITING FACTORS,

EXISTING PLANT LOADS AND PROJECTED PLANT LOADS

b) System Storage and WPCP Treatment for 1 overflow per year
(Storage = 700 ML)

- Base Flow, 2025 = 420 MLD
- Base Flow plus treat in 1 day (420 + 700) — 1120 MLD
- Base Flow plus treat in 3 days (420 + 230) — 650 MLD

4. Plant Expansion Requirements to Treat All Runoff Volumes (Storm Surge Stored in Sewer System)

a) System Storage and WPCP Treatment of 4 overflows per year
(Storage for average year = 620 ML)

- Base Flow plus treat in 1 day (420 + 620) — 1080 MLD
- Base Flow plus treat in 3 days (420 + 210) — 630 MLD
- Base Flow plus treat in 4 days (420 + 155) — 580 MLD

b) System Storage of all Storm Flows and WPCP Treatment of
1 overflow per year
(Storage for average year = 940 ML)

- Base Flow plus treat in 1 day (420 + 940) — 1360 MLD
- Base Flow plus treat in 3 days (420 + 310) — 730 MLD
- Base Flow plus treat in 4 days (420 + 235) — 660 MLD

storm were diverted to the plant. The interceptor sewer could possibly handle the instantaneous peak flow of the "As Is" 25 mm storm, but the high lift pumps could not handle the peak flow rate. This effect is worse for the "All Flow" 25 mm condition. In addition, much of these storms would need to be bypassed around the plant due to process limitations.

6.2.3 Flows To Be Treated with a Storage and Treatment Option

For a storage and treatment options, different amounts of combined sewage may be stored in storage tanks in the system. Two alternative storage volumes and options were examined in this study.

1. Storage in the system of only CSO volumes (i.e. storage of CSO's which are currently discharged to the receiving waters through the 22 outfalls). This storage option allows the storm surge to arrive at the WPCP. Equalization storage may, therefore, be necessary at the WPCP to control the storm surge as it arrives at the plant.

2. Storage in the system of volumes equivalent to all stormwater runoff (i.e. storage of CSO's (Item 1) and storage of all flows in excess of dry weather flows which enter the combined sewer system). Accordingly, the option does not provide equalization storage at the WPCP.

These alternative storage volumes will have different impacts on the WPCP. Basically, storage of only CSO's has a much larger impact on the performance of the plant than complete storage of stormwater inflows because high peak flow rates occur.

The flow rates to the WPCP from these alternative storage volumes are now described.

Storage of CSO Volumes Only (Storm Surge Directed to WPCP)

The option of storage of CSO volumes for 1 or 3 days and subsequent treatment result in the average daily flow rates next indicated in Table 6.6, Section 3. The average dry weather flow of 420 MLD is based upon a population of 500,000 targeted for the catchment area for 2025. A three day limitation on storage was used as a reasonable limit on storage since the average inter event period is of the order of 4 days.

Storage for a longer period (4 or 5 days) results in a small decrease in intercepted overflow volumes (see discussion in Appendix G). The flow rates to be treated at the plant are thus 920 MLD and

590 MLD for 1 and 3 days storage respectively, for the 4 overflow/year option, and 1,190 MLD and 680 MLD for 1 and 3 days storage for the 1 overflow/year option.

The CSO storage and treatment option for controlling CSO discharges to Hamilton Harbour thus is a much larger factor than the "limiting plant factors" summarized previously in Table 4.5. Using a traditional design rating approach, the plant capacity would need to be expanded by approximately 60 percent to handle storage of 500 ML fed the plant over three days (i.e. storage to handle 4 overflows/year) or by 80 percent to handle storage of 700 ML fed to the plant over three days. This assumes a conservative design capacity for the existing plant of 360 MLD (80 MIGD), rather than the "rated design capacity" of 409 MLD (90 MIGD).

Storage of All Stormwater Runoff in Sewer System (Storm Surge Stored in Sewer System)

The volumes to be stored for all stormwater runoff are next given in Section 4 of Table 6.6. They are approximately 20 percent larger than storage of only CSO volumes (compare for example storage requirements of CSO's: 500 ML versus all stormwater: 620 ML for the 4 overflow/year control strategy). The stormwater volumes could be stored and then diverted to the plant over a one, three or four day period. The flow rates to the plant are listed for these storage periods are given in Section 4 of Table 6.6.

Impact of Sewer System Storage Options on WPCP

Thus, whether to store all storm water or whether to store only CSO volumes and treat the remaining storm flows as it arrives at the WPCP results in four possible scenarios for the two control strategies of 4 overflow/year and 1 overflow/year. They are:

Scenario 1. Expand Plant to treat 4 overflows/year and store only CSO's in sewer system
- Storage – 500 ML
- Plant Treatment Scheme
- Day 1 540 MLD (possibly provide equalization storage of plant)
- Day 2 to 4 590 MLD (with 3 days storage in sewer system)

Scenario 2. Expand Plant to treat 4 overflows/year and store all stormflows in sewer system
- Storage – 620 ML
- Plant Treatment Scheme
- 630 MLD over 3 days
- 580 MLD over 4 days

Scenario 3. Expand plant to treat 1 overflow/year and store only CSO's in sewer system
- Storage – 700 ML
- Plant Treatment Scheme
- Day 1 660 MLD (420 + 240) (possibly provide equalization storage at plant)
- Day 2 to 4 650 MLD (with 3 days storage in sewer system)

Scenario 4. Expand plant to treat 1 overflow/year and store all stormflows in the sewer system
- Storage – 940 ML
- Plant Treatment Scheme
- 730 MLD over 3 days
- 660 MLD over 4 days.

The scenarios indicate that there is not too much difference in size of plant required when equivalent treatment periods are considered. For example, the control option for 4 overflows per year results in the following:

Store Only CSO's	Store All Storm Water
Day 1 – 540 MLD to treat diverted storm water	Day 1 to 4 – 580 MLD
Day 2–4 – 590 MLD	

For purposes of WPCP expansion studies, scenario 1 for the 4 overflow/year control strategy and scenario 3 for the 1 overflow/year control strategy were evaluated (i.e. storage of CSO volumes only in the sewer system and equalization storage at the WPCP, since equalization storage may provide flexibility for process control at the WPCP).

6.2.4 Summary of Limiting Factors for Woodward WPCP

Accordingly two sets of limiting factors were identified from this analysis.

- limiting factors of the present plant to handle small volumes of more frequent additional storm related flows
- limiting factors of the present plant to handle the large additional wastewater volumes due to population growth and full implementation of storage of CSO's to a target of 4 overflows/year or 1 overflow/year.

The limiting factors within the present WPCP, in order of priority, appear to be:

i) the hydraulic capacity and biochemical performance of the various units in the plant;
ii) the hydraulic capacity of various channels in the plant;
iii) the capacity of present high lift pumps to handle additional flows;

Limiting factors related to large additional flows are basically caused by insufficient capacity to treat these volumes. This was addressed by evaluating expansion requirements on average daily flow basis and then using simulation studies to provide somewhat more precise insights into expansion requirements.

6.3 Stormwater Runoff Analysis

6.3.1 Introduction

This section addresses the contributions of stormwater runoff to the Hamilton Harbour/Cootes Paradise area with the objectives of determining the relative contribution from each drainage area to loadings under both existing and future development conditions and determining the total loadings from stormwater systems.

The analysis for the separated stormwater system differs from the previous analysis in that the typical year was the only condition considered. The other considerations (i.e. design events and a dry/wet year) were not considered in this analysis since, unlike the combined sewer system and the WPCP, separated stormwater systems discharge directly to the receiving water bodies. That is, they currently have no components such as storage/treatment, which may significantly alter the loadings from the system. Therefore, the relative impacts of various sized events will not provide any additional insight on the response of the system.

6.3.2 Stormwater Runoff Volumes

Stormwater runoff volumes for existing and future development conditions were estimated for each major tributary discharging flows to Hamilton Harbour/Cootes Paradise. Existing and future development conditions in each of these drainage areas are listed in Table 6.7. Stormwater runoff volumes for the selected typical year (1989) are listed in Table 6.8. Key points as identified in the tables are:

1. Proposed developments within the area are concentrated in the Redhill Creek basin.

2. Overall, developed areas serviced by separated storm sewer services in the Hamilton Harbour/Cootes Paradise drainage basin are expected to increase by approximately 65 percent.

3. Proposed developments in the area are expected to increase stormwater runoff volumes by approximately 40 percent.

4. The Industrial Areas generate the largest contribution of urban stormwater runoff for existing conditions and the second largest volume for future conditions.

5. The Redhill Creek basin generates the largest contribution of urban stormwater runoff for proposed conditions. The second largest contribution for existing conditions is also from the Redhill Creek area.

Also note that while the separated sewer area increases by 66 percent under future conditions (Table 6.7) runoff volumes from these areas are anticipated to increase by approximately 40 percent. Runoff volumes do not increase to the same extent as drainage areas because:

- many of the anticipated developments (e.g. in Ancaster) are expected to be at a lower density than those which currently exist, and therefore runoff volumes per unit area will be less; and
- prior to development, the rural areas under consideration produced runoff during the May to October season.

TABLE 6.7

DEVELOPED AREAS FOR EXISTING AND FUTURE CONDITIONS IN THE HAMILTON HARBOUR/COOTES PARADISE DRAINAGE BASIN

Separated Storm Sewer Drainage Area	Developed Area (ha)*			Percent Increase In Developed Areas
	Existing	Proposed	Total	
Grindstone Creek	273	127	400	47
Spencer Creek	715	5	720	1
Rock Chapel Creek	160	320	480	200
Sulper/Ancaster Creek	1040	785	1825	75
Chedoke Creek	1340	140	1480	10
Redhill Creek	1760	2760	4520	157
Industrial Areas	950	–	950	0
TOTAL	6238	4137	10375	66

* Areas calculated using data on Figures 2.2 and 2.3.

TABLE 6.8

TYPICAL YEAR (1989) RUNOFF VOLUMES

FOR SEPARATED STORM SEWER DRAINAGE AREAS

Separated Storm Sewer Drainage Area	Existing Conditions		Future Conditions	
	Volume* (m^3)	Percent of Total Volume	Volume* (m^3)	Percent of Total Volume
Grindstone Creek	340,000	3	480,000	3
Spencer Creek	1,000,000	9	1,003,000	6
Rock Chapel Creek	250,000	2	575,000	4
Sulpher/Ancaster Creek	1,350,000	12	2,180,000	14
Chedoke Creek	1,900,000	17	2,060,000	13
Redhill Creek	2,630,000	27	5,400,000	39
Industrial Areas	3,400,000	30	3,400,000	21
TOTAL	10,870,000	100	15,098,000	100

* Includes volumes from existing separated sewer areas and volumes from rural areas slated for development as per the Regional Development Pattern (Figure 2.3). The volumes represent the quantity of stormwater which would be discharged from May 1 to October 31.

6.3.3 Stormwater Loadings

Stormwater loadings from the separated sewer areas for existing and future development conditions are listed in Table 6.9. Generally, with the proposed development patterns constituent loadings from separated sewer areas are expected to increase by approximately 40 percent.

6.4 Total Loadings to Hamilton Harbour/Cootes Paradise

Total constituent loadings to Hamilton Harbour/Cootes Paradise were estimated for flows originating from:

- CSO's
- WPCP discharges; and
- separated storm sewer discharges.

Both existing and future development conditions were included in the analysis. Future development conditions consider:

- a population of 500,000 is serviced by the Woodward Avenue WPCP; and
- future development will be concentrated in the areas specified in Figure 2.3.

Results are included in Table 6.10 and outlined below.

Under existing conditions:

- the WPCP is the major source of loadings of SS, TS, NH_3 TP, TKN, BOD_5 and Cu;
- CSO's are the main source of FC, and
- storm sewer discharges contribute the majority of Pb and Zn to the receiving waters.

With the projected population and development patterns:

- total loadings increase by approximately 30 percent;
- overall the WPCP remains the largest contributor of constituents to the receiving waters; and
- loadings from separated sewer areas generally equal or exceed loadings from combined sewer areas.

TABLE 6.9

SEPARATED STORM SEWER LOADINGS

TO HAMILTON HARBOUR/COOTES PARADISE

	Daily Loading (kg/day) for Specified Conditions		
Parameter	Existing	Future Conditions No Remedial Works	Percent Increase In Daily Loading
SS	6,000	8,200	37
TS	27,000	37,000	38
TP	20	27	35
NH_3	21	25	19
TKN	86	111	29
Zn	9	13	44
FC (total counts)	$1.4x10^{13}$	$2.10x10^{13}$	50
BOD_5	500	740	48
Pb	7.7	11	43
Cu	1.9	2.8	47

TABLE 6.10

TOTAL CONSTITUENT LOADINGS

TO HAMILTON HARBOUR/COOTES PARADISE

(MAY TO OCTOBER)

Parameter	Daily Loading (kg/d) for Existing Conditions				Daily Loading (kg/d) for Future Development Conditions			
	CSO	WPCP	Storm Sewers	Total (Rounded)	CSO	WPCP	Storm Sewers *	Total (Rounded)
SS	2,700	7,000	6,000	16,000	2,700	9,035	8,200	20,000
TS	8,500	150,000	27,000	186,000	8,500	195,000	37,000	240,000
TP	17	140	20	177	17	182	27	226
NH_3	30	2,086	21	2,140	30	2,710	25	2,765
TKN	76	3,058	86	3,220	76	3,975	111	4,160
Zn	7.9	3.1	9	20	7.9	4.0	13	25
FC (total counts)	$2.2x10^{13}$	$5.5x10^{11}$	$1.4x10^{13}$	$3.6x10^{13}$	$2.2x10^{13}$	$7.0x10^{11}$	$2.1x10^{13}$	$4.4x10^{13}$
BOD_5	410	2,800	500	3,700	405	3,610	740	4,755
Pb	3.3	2.1	7.7	13	3.3	2.7	11	17
Cu	0.9	6.3	1.9	9	0.9	8.2	2.8	12

* Includes loadings from existing separated sewer areas and loadings from rural areas to be developed as per the Regional Development Pattern (1989).

NOTE: See Appendix I for additional information with respect to the range of CSO loadings that may be expected within the Region.

In summary, the Woodward Avenue WPCP is the major contributor of constituents to Hamilton Harbour/Cootes Paradise, a condition that is expected to remain, assuming the effluent quality at the plant remains unchanged. The large contribution from the WPCP to the total constituent loadings is a result of:

- the continuous operation of the WPCP compared to the intermittent nature of CSO's (i.e. once every eight days or so);
- the higher concentrations of many of the water quality constituents in sanitary sewage; and
- the fact that the sanitary sewershed draining to the Woodward Avenue WPCP is approximately 20 percent larger than the combined sewershed under existing conditions increasing to about 100 percent under future development conditions, as it contains separated sewer areas such as Ancaster and Stoney Creek.

This observation suggests that although remedial works to reduce CSO's will substantially decrease CSO loadings, they will only decrease total loadings to the Harbour marginally, since CSO's are not the primary sources of most constituents. That is, substantial reduction in constituent loadings will require the improvement of the WPCP effluent quality. This issue is presented in Chapter 9, which outlines the effectiveness of the remedial works introduced in Chapters 7 and 8.

Based on the above three alternatives, the target level of reduction could be variable, i.e.: the level of reduction is dependent upon the sensitivity of the receiving body; or uniform if an end of pipe or BAT standard is selected.

The target level of reduction was discussed with the Technical Committee in the initial stages of the study. Based on the discussions, the following conclusions were drawn:

- The minimum target level should result in significant pollutant loading reductions (i.e. in the 50 to 60 percent range).

- Several target levels of reduction should be assessed to define the cost of meeting the targets and the relative cost effectiveness (i.e. does a moderate increase in the target level result in a substantial increase in cost to achieve this level).

- The draft documents as prepared for the Hamilton Harbour Remedial Action Plan (HHRAP) should be used to assist in establishing target levels of reduction.

- The target level should be defined based on frequency of overflows and pollutant loading reduction.

The HHRAP has been directed towards reducing loadings of nutrients, bacteria, metals and organics to Hamilton Harbour such that, in combination with control of other watershed areas (Spencer's Creek; Cootes Paradise; City of Burlington Stormwater and WPCP, Burlington Landfill Leachate), the following degree of water quality may be achieved ultimately:

- an oligotrophic–mestrophic state of eutrophication in Hamilton Harbour;
- a swimmable water at designated areas;
- a fishable water body with acceptable contaminant body burdens in fish where these body burdens originate from the harbour;
- a non–toxic set of discharges from all sources;
- water quality which does not pose a chronic toxicity hazard to biota;
- oxygen regimes which sustain a cool–water fishery in hypolimnetic waters.

After review of the HHRAP documents and a preliminary assessment of standards used elsewhere, the following three target levels were established:

- reduction of CSO's to four overflow per year (frequency standard);
- reduction of CSO's to one overflow per year (frequency standard);
- reduction of key water quality parameters to the levels as required in the report entitled "Preferred Options Report for Hamilton Harbour" (see Table 7.1).

The target levels would be applied to both the combined sewer system and to the Hamilton-Wentworth Sewage Treatment Plant.

In order to provide some insight as to what level of reduction the above targets would achieve, an initial analysis of the Strathearne Avenue sewer system was carried out (see Table 7.2). The following general conclusions were drawn:

1. Sizing CSO storage facilities using the proposed 4 CSO/year design storm (i.e. the 2-hour, 25 mm event) would reduce the existing CSO volumes to the receiving body of water (Hamilton Harbour) by 87 percent. CSO frequency would also be reduced from 27 to 4 events during a typical year.

2. Sizing CSO storage facilities using the proposed 1 CSO/year design storm (i.e. the 2-hour 35 mm event) would reduce the existing CSO volumes to the receiving body of water by 95 percent. CSO frequency would also be reduced from 27 to 1 event per year.

Based on the above findings, it was decided to deal with only the four overflows/year and one overflow/year standards as they bracketed the required levels of pollutant loading reduction as outlined in the RAP documents (i.e. 90 to 94 percent reduction).

With respect to defining the number of overflows/year, the period of May 1 to October 31 was used. Overflows may occur during winter periods if a rainfall event is combined with snowmelt. However, the frequency of overflows during this period is significantly less and furthermore, the technology and data required to model winter conditions were beyond the scope of this study.

7.3 Reduction Strategies for WPCP Discharges

Overall, the WPCP is the largest contributer of most constituents to the receiving waters. Substantial reductions in constituent loadings from the WPCP will therefore require improvements to the WPCP. However, it was decided with the Technical Committee that the objective of the WPCP component of

TABLE 7.1

Eutrophication Goals and Anticipated Conditions in Hamilton Harbour

Cootes Paradies and the Grindstone Creek Area

(from the Preferred Options Report for Hamilton Harbour)

Objective	Existing (1987)	Initial Goal (1990)	Final Goal (1995)	% Reduction for Final Goal
Phosphorus Loading (kg/day)				
Hamilton WPCP	290	140	60	79
Streams	95	90	65	32
CSO's	90	70	5	94
TKN Loading (kg/day)				
Hamilton WPCP	5300	2270	530	90
Streams	–	–	–	–
CSO's	200	160	20	90
Suspended Solids Loading (kg/day)				
Hamilton WPCP	3750	3750	900	76
Streams	31400	30000	20000	32
CSO's	2200	1400	200	91

TABLE 7.2

Preliminary Assessment of CSO Frequency and

Volume Reductions using CSO Storage Facilities

Conditions	Number of CSO Events	CSO Volume (m^3)*	Percent Reduction in CSO Volume
Existing	27	309,000	–
Proposed 1 CSO/Year Tank –With 3 day Drainage Rate	1	13,300	95
Proposed 4 CSO/Year Tank –With 3 Day Drainage Rate	4	39,700	87

* CSO Volume under existing conditions and CSO Storage Tank Overflow Volumes during a typical May 1 to October 31 period.

this study is to determine what remedial works are required at the WPCP to accompany any recommended CSO control works, in order to maintain, rather than improve the current effluent quality at the WPCP. Hence, no strategies to improve the effluent quality from the WPCP are proposed. Consequently, loadings from the WPCP are expected to increase under any recommended CSO control works which will direct larger volumes of sewage to the WPCP. The required WPCP works to achieve the loading reductions as specified by the Remedial Action Plan (RAP) for Hamilton Harbour are outside the objectives for this study.

Conversely, the WPCP does not contribute a substantial portion of the bacterial loadings (FC) to the receiving waters. High bacterial loadings, which tend to originate from CSO's (Table 6.10), are often responsible for beach closings in other areas of Ontario. Not suprisingly, the frequent closing of public beaches is often the driving force behind CSO controls.

7.4 Reduction Strategies for Separated Stormwater Loadings

Insight as to the overall level of reduction required for loadings from separated storm sewer areas may be obtained from the Preferred Options Report for Hamilton Harbour. The report addresses existing and final goals for loadings from streams in the area (Table 7.1). The stream drainage areas, however, include all urban and non-urban areas in the Region and surrounding areas, whereas this report addresses only those areas currently developed, and areas slated for development as illustrated in Figure 2.3. Therefore, loadings obtained in this study cannot be compared to those in Table 7.1 since the stream drainage areas differ. Rather, the stream data in Table 7.1 will be used to set a target level of reduction of approximately 30 percent for the combined loadings from existing development, proposed developments and remaining rural areas within the stream drainage basins.

Existing and proposed urban developments serviced by separated storm sewer systems in the area occupy approximately 14 and 9 percent of the total "stream" drainage area to Hamilton Harbour/Cootes Paradise, respectively. Obviously, in order to meet the required loading objectives in Table 7.1, loading reductions from the remaining rural area (i.e. farms, pasture, etc.) will be required. Furthermore, it is difficult to lower loadings from proposed development areas to below existing values since runoff volumes increase (i.e. usually by up to an order of magnitude) once an area develops. At best, loadings from newly developed areas may be limited to only marginal increases above pre-development loadings using Best Management Practices. This study will, therefore, assume:

- loadings from existing separated storm sewer services will remain "as is";
- loadings from areas slated for development are to be kept as close to existing levels as possible using Best Management Practices; and
- loadings from remaining rural lands (i.e. farms) are to be reduced in excess of 30 percent in order to meet the loading objective set forth by RAP.

The latter point will have to be addressed in future studies as the reduction of constituent loadings from non-urban areas is beyond the scope of the study. The required WPCP works to achieve the loading reductions as specified by the Remedial Action Plan (RAP) for Hamilton Harbour are aslo outside the objectives for this study.

8 - IDENTIFICATION AND SCREENING OF REMEDIAL ALTERNATIVES

8.1 General

The evaluation of remedial alternatives must consider many factors. For example, technical feasibility and cost are two factors which must be considered. Additional factors include operation and maintenance, ease of implementation, policy development and public/municipal acceptance.

The evaluation of remedial alternatives in this study was carried out in two phases. The first phase involved an evaluation of all potential alternatives. The alternatives was evaluated based on the following factors:

- technical feasibility;
- cost-effectiveness;
- ability to meet the desired objectives.
- implementation considerations;
- municipal/public acceptance
- conflict potential
- environmental assessment process
- operation/maintenance considerations

Based on these factors each alternative was given a rating of good, fair or poor. The good and fair alternatives were then be looked at in more detail.

Table 8.1 lists the remedial alternatives which were initially evaluated. The alternatives have been broken down into the following categories:

- source control;
- sewer system control;
- wastewater treatment strategies.

The following section provides an overview for each remedial alternative which was considered

TABLE 8.1

LIST OF PRELIMINARY REMEDIAL ALTERNATIVES

TYPE	REMEDIAL ALTERNATIVE
Source Control	Street Cleaning
	Public Education Programs
	Spill Prevention and Management
	Sewer Use By-Law Enforcement
	Sanitary System MISA Program
	Roof Downspout Disconnection Program
	Infrastructure Rehabilitation
	Sewer Flushing
Sewer System Control	Adjustment of Overflow Regulators
	Swirl Concentrators
	Sewer Separation
	Inlet Control Method
	Real Time Control
	Inline Storage of CSO's
	Offline Storage of CSO's
Wastewater Treatment Strategies	Improved Treatment at Present Woodward WPCP
	Expansion of Biological Processes at Woodward WPCP
	Step Feed at Dundas WPCP
	Alternative Phys-chem Technolgoics at Woodward WPCP
	Small Phys-chem Plants or Coarse Roughing Filters Adjacent to Sewer System Storage
	Engineered Wetlands

8.2 Description of Remedial Alternatives

8.2.1 Source Control

Street Cleaning

Street cleaning (or sweeping) is a practice commonly undertaken by municipalities to clean accumulated sediment and debris from the streets. Common methods of street cleaning are manual clean-up, manual or mechanical broom sweepers, vacuum sweepers and street flushing. In theory, these practices may be an effective method for reducing pollutant loadings as they transport pollutants from the streets into the sewers during dry weather when treatment is available.

Results from the Toronto Area Watershed Management Strategy (TAWMS 1986) and U.S. National Urban Runoff Program (U.S. EPA 1983) suggest that street cleaning is generally ineffective in reducing the overall pollutant loading. In typical municipal programs with sweeping frequencies of once or twice per month, the removal efficiency for suspended solids was less than 5 percent. The EPA study also concluded that street cleaning is not an effective method for removing organic contaminants, nutrients and/or coliforms.

All municipalities within the Region do carry out street cleaning programs. Specifics of the City of Hamilton program are given below.

1. Sweeping commences from April 1 to November 15 each year.

2. Streets with alternate parking are swept once a month.

3. Main and secondary streets are swept once a week.

4. Streets with no parking restrictions are hand posted and swept three times a year.

5. During the winter months, streets are swept when the temperature is above 3 degrees celsius.

6. All residential streets are swept once a month.

All streets are mechanically broom swept and then power flushed. The annual budget for carrying the above program (1989 dollars) may be broken down as follows:

- Mechanical Street Sweeping $689,560.00 ($14.73/pass kilometre)
- Mechanical Street Flushing $252,629.00 ($13.19/pass kilometre).

Based on the results of the TAWMS and EPA studies, it is not recommended that this program be enhanced in order to reduce pollutant loadings.

Public Education Programs

Public education or awareness programs involve preparation and dissemination of information regarding practices which can be undertaken to improve overall water quality. Information on specific practices can be passed on to the public through brochure, information booths/centres, advertisements in the local media, and special public information events. Typical issues addressed include:

- pet litter control;
- general litter control;
- application of lawn and garden chemicals;
- spill prevention and management;
- management of hazardous household waste; and
- solid waste management/disposal.

The Region currently has in place manual litter pick-up in the City of Hamilton core and actively encourages dog owners to clean up after their pets in active parks.

Public education programs may not result in substantial pollutant loading reduction to the receiving waters, however, the programs are generally cost-effective and may be used to provide insight as to how each individual may assist in improving the quality of the environment. For this reason, public education programs should be continued or enhanced.

Spill Prevention and Management

Spill prevention and management provides both a means of attempting to minimize potential for spills and an efficient manner of addressing incidents when they occur. Currently, the Ministry of Environment has in place a Spills Response Program. The Ministry of Environment under the Environmental Protection Act (Part IX) has the provisions to act as regulatory agency to enforce duties on persons responsible for spills, recommended cleanup procedures, and evaluate the adequacy of cleanup and disposal efforts. The primary problem this type of alternative is "shock" loadings of pollutants to receiving waters. The Ministry of Environment program is not for prevention, but rather for response to spills.

Measures to minimize the impact of accidental spill of contaminants including provision of underground oil/grit separators at commercial and industrial developments and buffer strips between storm sewer outfalls and receiving waters. Oil/grit separators are commercially available or can be modified reinforced precast concrete vaults. To be effective, they should be a functioning component of the storm sewer system and be located at the property line of development sites to allow for inspection and maintenance by municipal staff.

The cost for oil/grit separators could be borne by future development with implementation made a condition of Draft Plan approval and the Subdivision Agreement. This type of measure could be applied both to new development sites and existing areas as they come up for re–development.

Buffer strips between storm sewer outlets and receiving waters provide a containment area for accidental spills. Management measures for prevention of spills or techniques for spill containment prior to entry into the natural environment warrants consideration for implementation.

The Region has an emergency plan to deal with spills resulting from events such as traffic accidents to contain the spreading of a spill. The Ministry of the Environment is also involved in the program. Spills to the harbour also involve the Harbour Commission.

Spills do have an adverse impact on the environment if they are not appropriately contained. For this reason, a Spills Prevention and Management Program should be encouraged. The impact of the shock loading of a spill (within a CSO) is, however, difficult to ascertain due to the limited available information and the random nature of spills.

Sewer Use By-Law Enforcement

Sewer Use By-Laws are municipal by-laws for regulating discharges to sanitary and storm sewers. These by-laws control the discharge of several parameters which include bacteria, nutrients, solids and heavy metals. A key factor is that sewer use by-laws govern discharge of parameters to and not from either sanitary or storm sewers. In general, violation of sewer use by-laws are primarily from industrial sources and impact dry weather conditions.

The Region has recently adopted a sewer use by-law which is quite similar to the Provincial Model Sewer Use By-law.

Implementation of programs of this nature should be encouraged as they reduce pollutant loadings during both dry and wet weather periods.

Source Control of Sanitary Sewer System Under MISA Program

Industrial discharges of metals and other organics to the sanitary sewer system/combined sewer system cause these substances to reach receiving waters such as Hamilton Harbour when CSO's occur. Even if these wastes reach the wastewater treatment plant, they are not removed as effectively as new effluent quality criteria may require.

Based upon the low degree of treatment provided by wastewater treatment plants in the Los Angeles area, a "source control" program was instituted, directed towards decreasing/removing these discharges to the sanitary sewer system by providing on-site industrial treatment prior to discharge of the treatment effluent to the sanitary/combined sewer system. Monitoring data suggests that the influent loading to these treatment plants have decreased approximately an order of magnitude over the past decade with the implementation of "source control".

The MISA program is presently evaluating the sources and contaminant levels of industrial discharges to the Hamilton-Wentworth Sanitary/Combined Sewer system, preparatory to assigning discharge limits to the various industrial discharges. Upon completion of these studies and implementation of methods for effecting a reduction, a substantive reduction in discharges of these contaminants through CSO's and wastewater treatment discharges to Hamilton Harbour can be anticipated. The reduction in releases to the sewer system may take the form of lower concentrations (by using on-site treatment) and lower flow rates released by the industries (by lowering water consumption and/or recycling).

Roof Downspout Disconnection

Presently, a majority of the roof downspouts within the study area are connected directly to the storm or combined sewer. Disconnection of the roof leaders may lead to a reduction in flow and pollutant loadings to the sewer system.

Several factors must be considered prior to carrying out a program of this type. These include:

- public willingness to co-operate;
- existing by-laws;
- liability considerations;
- effectiveness.

The City of York has recently completed a pilot project whereby voluntary disconnection of downspouts was carried out. The area under consideration had relatively small lots (35 feet by 110 feet). Furthermore, the grading around the homes was not consistent.

The program was voluntary in nature and all work was carried out by the Public Works Department. The objective of the program was to reduce flows within an area which is subject to basement flooding. The Works Department prepared a flyer outlining the benefits of disconnecting downspouts and provided, at no cost to the homeowner, the labour and materials.

The results of the program were such that about one in three to one in four downspouts were disconnected. Of the downspouts which were disconnected, slightly more than half discharged to grassed areas (the remaining percentage discharged to driveways). The estimated cost per downspout disconnection, including labour, was $100 (updated to 1990 dollars).

Based on the results of the City of York study and the existing lot grading and lot size within a majority of the study area the anticipated effectiveness of a roof downspout disconnection program would be mixed, at best. Furthermore, a widespread program would be relatively expensive (the estimated cost is $15,000,000 if each residential downspout is disconnected).

An quantitative assessment of the effect of roof downspout disconnection on CSO frequencies and volumes is included in Section 9.3.2.

Infrastructure Rehabilitation

The primary target for this alternative would be the reduction of infiltration/inflow. This, in turn, would reduce the volume of CSO into the receiving waters.

Excess infiltration/inflow may be reduced by:

- rehabilitating the sewers located within public and/or private property;
- maintaining (or periodically checking) structures such as control gates to Lake Ontario to ensure lake water does not enter the system.
- minimizing leakage from the water supply system that infiltrates into the combined or storm sewer systems

The flow monitoring carried out in this study would suggest that the infiltration/inflow within the system is not excessive. Furthermore, rehabilitation of the sewer systems solely to reduce CSO's would likely be an ineffective and costly alternative. Thus, this alternative does not need to be considered further.

Conversely, field visits carried out during the course of this study showed that several control structures were not functioning properly. This may result in Lake water infiltrating back into the combined sewer system or inadvertent CSO's. A regular maintenance program for all control structures should therefore be carried out.

Sewer Flushing

Wastewater solids which settle within the combined sewer during dry weather conditions and are then re-suspended during wet weather conditions may contribute substantial loadings during a rainfall event. Studies in Buffalo, New York (Field, 1972) have shown that 20 to 30 percent of the annual collection of domestic wastewater solids settle and are eventually discharged during storm events. Solids buildup within the Hamilton combined sewer system was noted during the flow monitoring program at Strathearne where up to 25 mm of solids had accumulated during dry weather conditions. The material seemed to be mobile in that is was washed out during severe rainfall events.

The effectiveness of sewer flushing has not been extensively tested. A research project by U.S. EPA (Pisano, 1979) concluded that small volume flushing would transport organics, nutrients and heavy metals thus making the option feasible and attractive.

Sewer flushing of large diameter combined sewers was investigated for the City of Elizabeth New Jersey (Bogert Assoc., 1981). The preliminary findings (evaluation of the system is still ongoing) showed that an automated flushing system reduced the pollutant load within problem areas by 25 to 30 percent. The diameters of the sewers ranged from 460 mm to 1400 mm.

The results of the above noted studies suggested that the cost of installing equipment to flush sewers is roughly equivalent to other structural alternatives (eg: storage tanks). Furthermore, operation and maintenance costs will be substantial. In light of restricted municipal budgeting and staffing allocations and the limited amount of available data this alternative is not recommended for further evaluation.

8.2.2 Sewer System Control

Adjustment of Overflow Regulators

The frequency of overflows at a given location is dependant upon several factors including drainage area, percent sewer separation, and the type and setting of the overflow regulator. The frequency and volume of overflow at a given location may therefore be varied by adjusting or reconstructing the overflow regulator.

Adjustment of overflow regulators to reduce CSO's at selected areas (i.e. areas which are sensitive to CSO's) would be of benefit in that not only would the annual volume of overflow be reduced but the associated nuisance problems related to floatables and odours could be minimized. To compensate for the increased hydraulic loading within the system due to reducing the overflow at one or more locations it may be necessary to increase the frequency and volume of overflows at another location. Otherwise, the loading to the WPCP will increase (leading to the possibility of increased overflows at the WPCP).

A field inspection was carried out during this study to confirm the type and setting of each major overflow regulator (see Section 4). Results of this inspection, together with some straightforward hydraulic calculations showed that the level or frequency of overflows within the system would vary significantly, due to the setting or type of each regulator. For example, the regulator upstream of the Sterling outfall can convey flows up to 2.3 l/sec/ha of impervious area, prior to overflowing. Conversely, the regulator at Queenston Road can convey flows up to 80 l/sec/ha of impervious area, prior to overflowing.

The results as provided in Figure 4.1 would also suggest that the highest settings for the regulators are not always at the overflow locations adjacent to areas which are particularly sensitive to overflows.

The ultimate remedial plan will result in a significant reduction in the number of overflows at each of the 22 sites. However, based on existing availability of funding the ultimate plan will take approximately 20 years to complete. It is therefore worthwhile to look at alternatives such as adjustment of overflow regulators as these types of alternatives are cost effective, significant with respect to improving conditions at key locations and are relatively easy to implement (for example, many of the existing regulating devices can be adjusted by building up a weir or adjusting a sluice gate).

Overflow regulator adjustment as a remedial option is discussed in detail in Section 9.

Swirl Concentrators

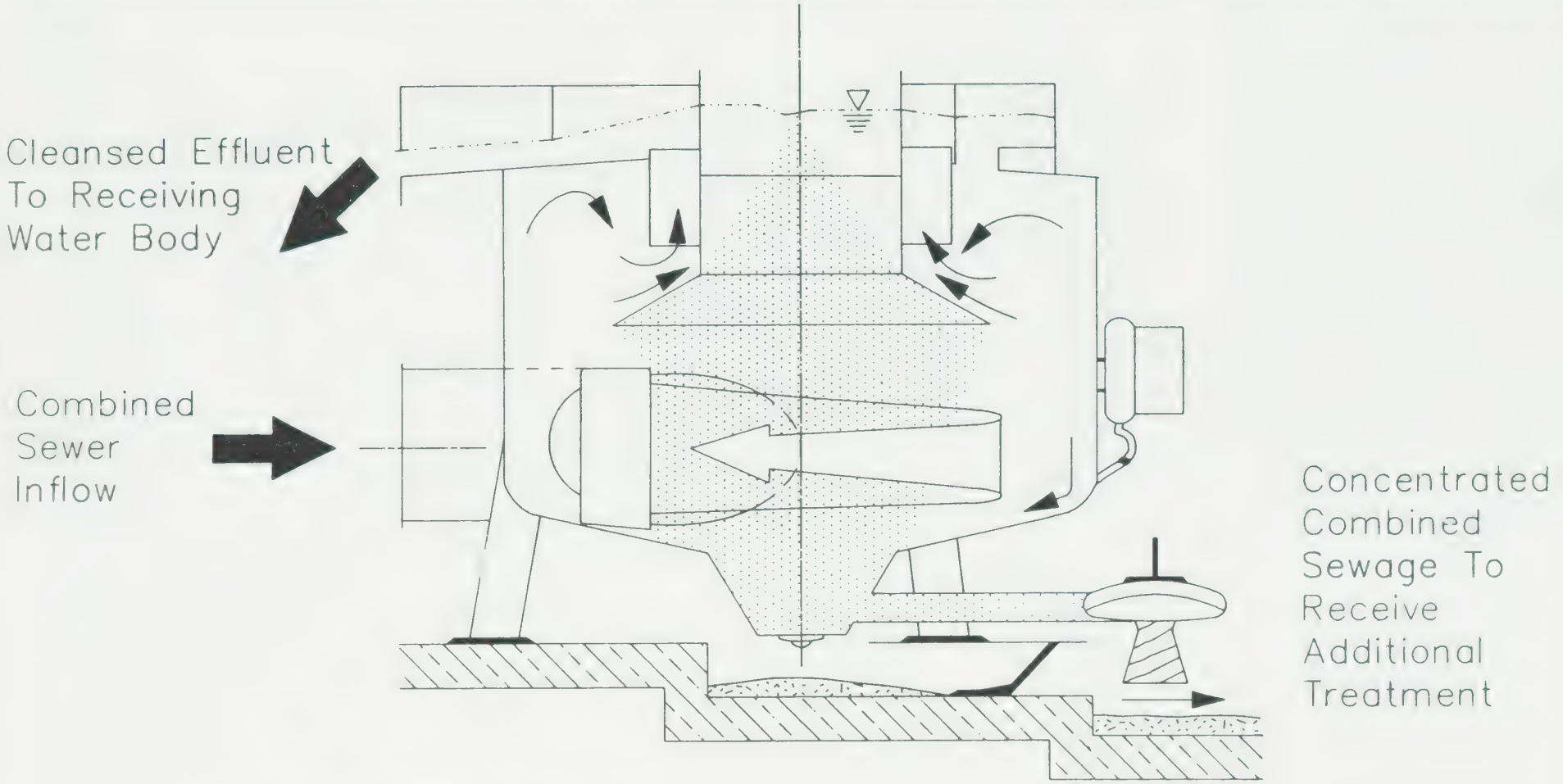
The swirl concentrator (Figure 8.1) is a small, compact solids separation device with no moving parts. During periods of high flow the outflow from the facility is throttled. This results in the facility filling up, and to self induce a swirling vortex-like operation. In theory, the concentrated foul matter is intercepted for treatment while the cleaner, treated flow discharges to the receiving body.

The facilities are designed to remove heavy organic and inorganic material such as sand and gravel. Floatable removal is generally also achieved.

There are many swirl concentrators (or similar versions) within the United States, England and Germany. The initial devices constructed in the United States were adaptations of an English design developed in the early 1960's (Smisson, 1967). Recently, improvements to the initial design have been made by English (Smisson, B) and German (Brombach, H.) investigators.

The effectiveness of the facility is a function of several parameters including:

- size of facility
- flow rate through facility
- particle size gradation of influent
- specific gravity of particles
- detention time.



SWIRL CONCENTRATOR LAYOUT

N.T.S.

SOURCE: Water Pollution Control Federation, Manual of Practice No. FD-17,1989

Figure 8-1

The swirl concentrator may be a cost effective alternative to treating CSO's as the size of the facility will be considerably smaller as compared to a storage tank.

The results, to date, have however, been mixed with removal rates of suspended solids varying between 20 to 50 percent depending upon the parameters listed above. Furthermore, the minimum diameter of particle to be successfully removed from the effluent ranges between 40 to 100 microns.

Recent designs in the United States have combined the swirl concentrator with other technologies. Figure 8.2 illustrates one installation in Decatur, Illinois (Hunsinger, 1987). In this installation the swirl concentrator was used together with a screen chamber, and first flush storage tank. Collectively the system is designed for the 1 year, 1 hour storm event.

Based on the treatment efficiencies quoted in the literature and discussions with suppliers/manufacturers of the technology combined with the pollutant removal standards as outlined in the HHRAP documents the swirl concentrator as a stand alone technology would not seem to be feasible. The swirl concentrator, together with a small tank to capture flows from the more frequent events and other technologies such as grit chambers or disinfection may, however, prove to be effective. This technology will not be considered further in this study as the results with respect to treatment efficiency are not conclusive. It should, however, be considered in the future if pending results prove to be positive and the cost effectiveness is confirmed.

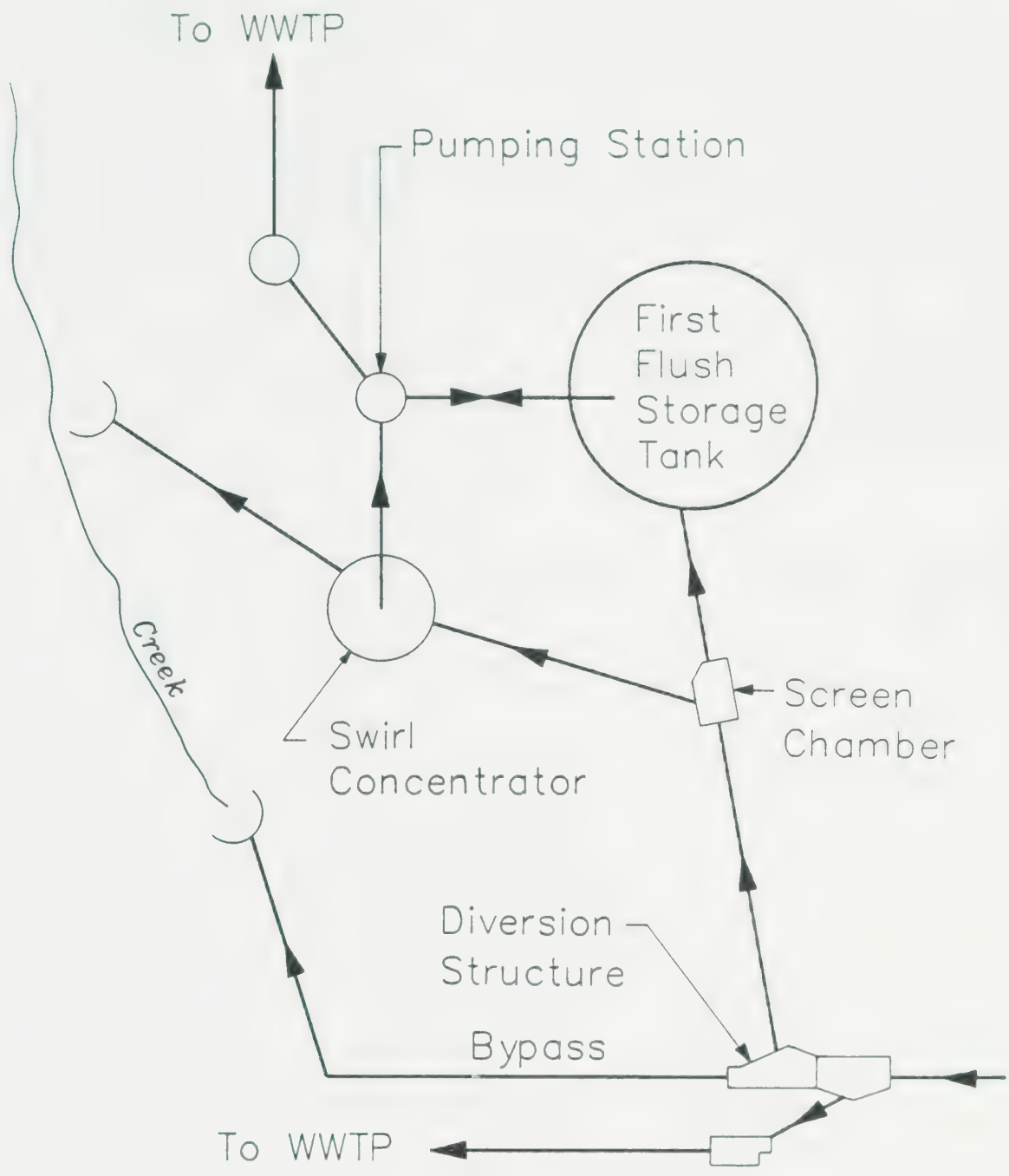
Sewer Separation

For may years sewer separation was viewed as the preferred alternative for reducing combined sewer overflows and thereby improving water quality. However, in recent years the trend has been to move away from sewer separation programs for the following reasons:

- cost
- time required to complete the separation program
- effectiveness
- impact on water quality

Recent studies have shown that other alternatives are more cost effective than sewer separation. Furthermore, remedial alternatives such as end of the line storage facilities can be constructed relatively quickly as compared to storm trunk sewers and the associated lateral sewers along local streets.

Figure 8·2



STORAGE FACILITY AND SWIRL CONCENTRATOR ARRANGEMENT

N.T.S.

SOURCE: Water Pollution Control Federation, Manual of Practice No. FD-17,1989

The effectiveness of a sewer separation program is also limited in that it is generally not feasible to separate roof drains or catchbasins located on private property (see previous section). Combined sewer overflows will, therefore, likely occur after the sewer separation program on public property has been completed.

Lastly, the impact on water quality, due to separation programs has been defined in recent studies (Don River and City of Sarnia). The results of these studies show that separation programs generally provides only a marginal reduction in pollutant loading. In some cases the loadings of some water quality parameters actually increased.

In light of recent findings, sewer separation, as an alternative for improving water quality should not be considered further. Sewer separation, on a local basis, may be required, or cost-effective to alleviate other problems such as basement flooding.

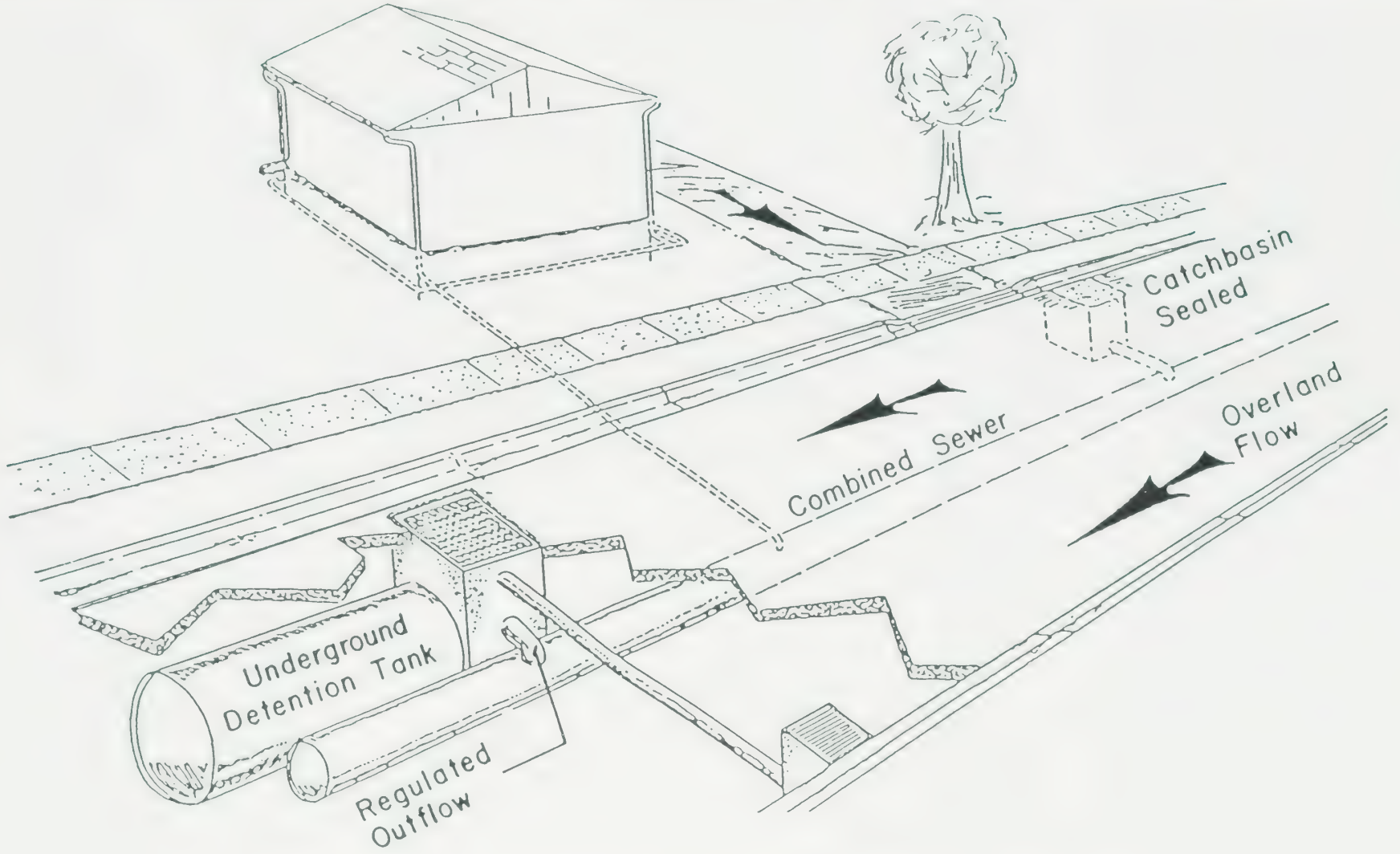
Inlet Control Method

The Inlet Control Method is illustrated in Figure 8.3. Flows which would have normally entered the sewer system uncontrolled are throttled or controlled. Excess flows are then directed overland to low lying areas. Underground storage facilities are then constructed to store the excess flows.

The basic concept of the Inlet Control Method is to restrict, for any level of storm, the inflow to the sewer system such that excessive surcharging and associated basement flooding does not occur. The Inlet Control Method has been used to reduce basement flooding problems in numerous municipalities within Ontario (i.e. City of York, City of Sarnia and City of Windsor).

An assessment to define the effectiveness of the Inlet Control Method in reducing combined sewer overflow volumes was carried out in the Humber River Water Quality Management Plan (TAWMS, 1986). The findings suggested that the local underground tanks which were required to reduce basement flooding had minimal impact on CSO volumes.

Based on these findings, the Inlet Control Method, as a stand alone alternative for reducing CSO's will not be considered further. The Inlet Control Method, on a localized basis to alleviate basement flooding may well prove to be cost-effective.



INLET CONTROL CONCEPT WITH UNDERGROUND DETENTION SYSTEM

Figure 8·3

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Off-Line Storage Facilities

Off-line, or end of the line storage facilities, generally involve the storage of overflows at or near the point of discharge (see Figure 8.4). This structural alternative is commonly used in various jurisdictions throughout North America and Europe (there are an estimated 10,000 storage facilities within Germany).

In theory, off-line storage can be located above or below ground level. In practice the facilities are generally located below ground to reduce potential odour problems. The inflow to the facility is controlled by the overflow device within the combined sewer system. The overflow may be controlled by a weir, orifice, gate or pump.

A properly designed facility should consider the following:

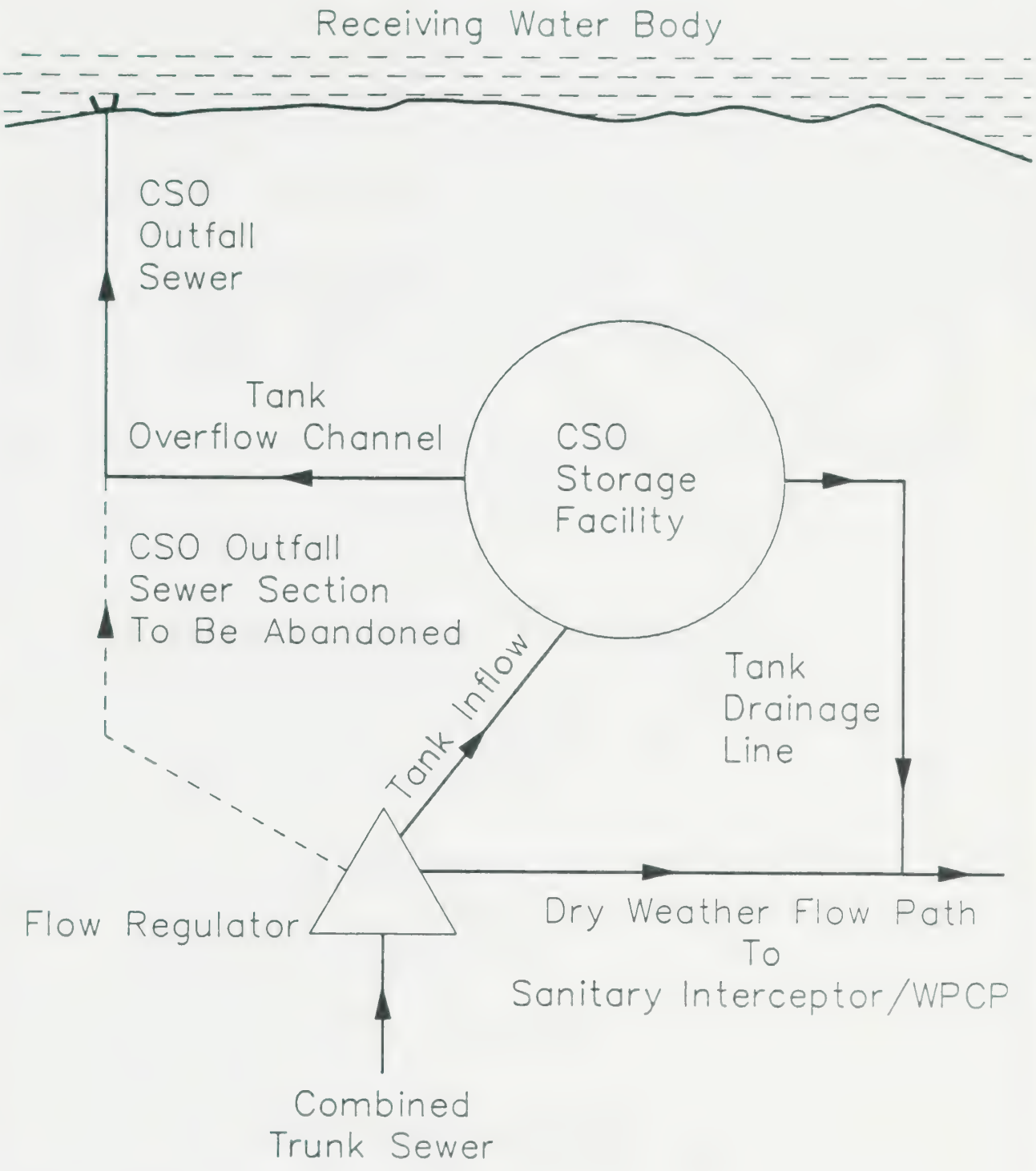
- odour control
- access
- settling of solids
- capture of floatables
- ventilation
- cleaning of facility

A practical limitation of using off-line storage facilities involves the availability of land, especially in built-up urban areas. In the cases where land availability is a problem it will generally be necessary to link several overflows together by constructing a relief sewer which, in turn, conveys flows to a central facility.

The option of constructing off-line facilities is frequently compared to the option of constructing in-line facilities (see next section). Generally, other studies (Roesener, 1989) have shown that the larger the storage volume the more cost-effective inline storage becomes, especially if tunnelling is required.

In general terms, the above would suggest that end of the line storage is likely to be more cost-effective for storage of smaller, more frequent events. Inline storage will tend to approach, on a unit basis, the cost associated with offline storage for the larger, less frequent events.

Figure 8·4



TYPICAL OFFLINE CSO STORAGE FACILITY

N.T.S.

In summary, construction of offline facilities to store CSO's is generally found to be a technically feasible and cost-effective alternative. This option will be evaluated further in this study.

In-Line Storage

In-line storage generally involves oversizing of existing sewers, twinning existing sewers or constructing in-line basins. The storage may be controlled manually by constructing weirs, orifices or gates or automatically using gates, pumps etc.

In-line storage for larger systems is commonly provided by constructing a tunnel sewer, well below the surface. In-line storage has the advantage over off-line storage in that land requirements are usually kept to a minimum if existing easements are utilized. Furthermore, disruption to homeowners is generally minimized if a tunnel is constructed.

In-line storage may also be beneficial in situations where an existing trunk sewer is undersized, or provides the only alternative for conveying flows from a large section of serviced area (the later case occurs within the Region as the Burlington Street interceptor services a majority of the Central Business District). In this case, twinning the existing sewer to improve the level of service and reduce CSO's may be provided by the additional interceptor.

Construction of in-line storage facilities must consider several factors including:

- buildup of toxic or hazardous gases
- access and safety
- provision of self cleaning velocities during dry weather or low flow conditions
- energy dissipation within drop shafts

In-line facilities will be considered in further detail within this study.

Real Time Control

A Real Time Control System involves the collection and dissemination of data using an automated or computerized system. The data which is collected is evaluated in a real time mode in order to achieve better performance. Generally this is achieved by automatically adjusting gate positions, pump speeds

etc. during the storm event such that the use of storage and conveyance within the system is maximized. Many Real Time Control Systems make predictions of flow volumes (and thus operational patterns) based on cumulative rainfall amounts.

The major components of a Real Time Control System generally includes a real time monitoring network and telemetry system, a micro–computer controller for the monitoring network and control structures and computer software and hardware. Operation may be performed locally by the operator, under supervisory control of the operator and/or automatically by the computer.

As the systems are automated operation and maintenance is generally relatively straightforward. A scheduled or preventative maintenance program is, however, generally required to ensure that all components function satisfactorily.

One of the primary advantages of a Real Time Control System is that the system accounts for the dynamic, spatial and temporal properties of rainfall events. Without adequate knowledge of these properties gates or valves may be opened prematurely resulting in overflows at certain locations while other components of the system are under–utilized.

There are several Real Time Control Systems installed within the United States and Europe including systems in the Cities of San Francisco and Seattle.

The Hamilton–Wentworth Regional Engineering Department, in conjunction with the Ontario Ministry of the Environment is currently conducting a demonstration project to evaluate micro–computer based Real Time Control Systems for managing CSO diversion structures (Robinson, Stirrup, 1990). The Royal Avenue diversion structure has been selected as the demonstration site.

The objectives of the study are to:

- test the feasibility of this new technology;
- evaluate the Real Time Control System with respect to its ability to reduce CSO's to the local receiving waters.

Based on the performance of the proposed Real Time Control System, a theoretical operating strategy for a larger portion of Hamilton's trunk sanitary interceptor will be developed.

In summary, Real Time Control Systems have been installed and are utilized within several large municipalities worldwide. The systems would seem to be especially attractive in cases where rainfall patterns are diverse enough to result in an unbalanced loading within the system and where the opportunity exists to convey or transfer flows from one are to another.

The rainfall patterns within the Region, due to existing topographic features and wind directions do result in situations where areas within the system experience significant runoff volumes while other areas are subject to relatively low rainfall and runoff volumes. Furthermore, the existing system does have spare capacity during the more frequent events. This would suggest that the evaluation of a Real Time Control System would be a viable option for reducing CSO volumes and frequency.

As one of the mandates within an ongoing study (Region – Ministry of the Environment) is to evaluate the effectiveness of a Real Time Control System, this option will not be further evaluated in this study.

8.2.3 Wastewater Treatment Strategies

Improved Treatment in the Present Woodward WPCP

Special options for improving treatment include:

- enhance settling performance on the north side of the Woodward Avenue WPCP.
- other process modifications.
- step feed

Step feed control is used to minimize solids loss during storm flows. It is useful for municipal wastewater treatment facilities which are frequently subject to rapid and sustained increases in flow rate caused by the entry of storm water into the sewer system.

Because conventional activated sludge plants have limited hydraulic dampening capacity, high flow rates transfer additional solids from the aeration basin to the secondary settler and increase the settler sludge blanket depth. Solids washout occurs when the top of the blanket reaches the effluent weirs. To prevent washout and retain biological solids, many plants must bypass flows during storms, discharging untreated sewage into the environment.

Step feed can be adapted to activated sludge systems with tanks-in-series or plug flow aeration basins. The MLSS concentration increases in the zone created upstream of the point of addition while the concentration decreases in the contact zone downstream. Consequently, the solids loading to the final clarifier is reduced and the potential for solids washout lessened.

The point of influent addition is manipulated during step feed operation to control the solids distribution within the aeration basin. The closer the point of addition to the end of the aeration tank, the greater the reduction in suspended solids concentration in the feed to the final settler. However, shifting the feed point along the tank also reduces the effective aeration volume available for treatment of soluble organics.

Dundas WPCP

Due to the success of step feed work, one option is to optimize use of the Dundas WPCP to handle stormwater flows. This would require the continuing use of the Dundas plant as a normal operating treatment facility in order to have well conditioned activated sludge available for treating storming water whenever the storm occurs.

Decisions upon the future of the Dundas WPCP have not yet been made, in part because the effectiveness of step feed there has only recently been documented. Accordingly, the role of the Dundas WPCP in a Pollution Control Plan should be further evaluated in future studies.

Expansion of the Woodward Avenue WPCP

After review of the possible improved performance which would be attained at this present Woodward Avenue plant, it is clear that it cannot treat much additional storm flows. Accordingly plant expansion alternatives were evaluated. This involved evaluating the hydraulic capacity of various existing connecting channels, and a process evaluation of the primary clarifiers, the secondary aeration tanks, and the secondary clarifiers using the process simulation model.

Other Wastewater/Stormwater Treatment Devices

Three other Treatment Devices were examined in this study for wastewater/combined sewage treatment:

(i) Small Phys–Chem Plants (Coagulation – Flocculation – Settling)

(ii) Engineered Wetlands

(iii) Coarse Roughing Filters (CRFs)

These options were considered in brief detail, but require further analyses or pilot scale testing prior to evaluating their usefulness for purposes of the Pollution Control Plan. A fourth option for treatment of separation storm sewer runoff – passive wet ponds was also overviewed.

Small phys–chem plants, engineered wetlands, and CRF's were evaluated as a partial substitute for the storage and treatment option to handle the impacts of stored CSO's. If used as a partial substitute for the storage and treatment option, they would be built in the combined sewer system near Chedoke Creek, Cootes Paradise, Hamilton Harbour or Red Hill Creek. Dependent upon the option, they could be a partial or total substitute for storing volumes of CSO's.

Small phys–chem plants, consisting of coagulation–flocculation–settling basins would be used to treat combined sewer overflows as they occur, at specific outlets. The technology is widely used in water treatment plants and hence is well understood at a process level. It should be able to attain substantial removal of suspended solids, contaminants associated with suspended solids, and should coprecipitate metals and phosphorus. It, however, requires further evaluation, prior to considering it as a central element in a pollution control plan because of uncertainties associated with the performance of the sedimentation portion of it under the dynamics of storm water hydrographs.

A possible substitute technology for the "phys–chem" plants is the coarse roughing filer. It has received extensive application in developing countries to treat highly turbid water (up to 1500 – 2000 mg/L of suspended solids). It consists of the addition of a metal coagulant (such as alum, ferric chloride, or polyaluminum chloride) to the water requiring treatment and then direct filtration of the coagulated water. The filter media is not conventional. The media is quite large (grain size ranging from 2 cm to 6 cm), resulting in large void spaces for accumulating the flocculated suspended solids, but allowing high filtration rates and small reduction in head loss. Its advantage in comparison to the small phys–chem plants is that the potential upsets of clarification do not result.

This technology requires pilot scale work to evaluate performance, operation and maintenance considerations (e.g., bacterial growths during interevent period; backwashing once a month or once per season) before it can be considered as a central element in a pollution control plan.

Engineered wetlands are wetlands in which partially treated flows are pumped to a ponded area (0.1 – 0.4 m deep) where cattails and other plants grow and where polishing of the water to remove BOD, suspended solids and some forms of nitrogen occur through natural processes. The availability of suitable tracts of land near the wastewater treatment plant preclude the usage this option to any major extent.

Water Recycling

The Woodward Avenue plant receives wastewater from a municipality with one of the highest per capita usages in Ontario (approximately 900 l/person/day). Additional water cycling by the major users due to active water usage management programs or due to MISA related evaluations for surcharges to sanitary systems would reduce flows to the plant (a reduction by 25% would decrease the plant inflow by approximately 100 MLD for a population of 500,000).

Education Programs

Education programs will assist in reducing contaminant addition to stormwater Priorities could be targeted to herbicide, pesticide, and fertilizer use in separated stormwater areas which will receive little treatment as a priority measure. These areas include Chedoke Creek on the mountain above the Golf Course, Red Hill Creek above Stone Church Road, and Stoney Creek below the mountain.

Wet Ponds

For treatment of runoff in existing or new urban areas served by separated storm sewers, wet ponds or equivalent technology are a passive device which remove contaminants from stormwater runoff by settling solids and chemicals (phosphorus, metals) attached to the solids. When combined with social (recreation) and environmental (wetlands) amenities, these BMP's achieve extensive water quality control (80 percent removal of SS, 50 percent of TP) and enhance the aesthetics of the surrounding areas.

There are few potential locations for the ponds left in the Hamilton–Wentworth Region in the Red Hill Creek and Chedoke Creek areas where they would receive drainage from large enough watersheds (100 ha), since these watersheds have developed before suitable locations were set aside for this technology. Hence they will not have a large effect for purposes of this PCP. As the plan developed

in this study is implemented, it is suggested planning efforts be further directed towards examining potential areas for building wet ponds or equivalent technology in the area of Red Hill Creek just above the mountain brow, and in the area of Chedoke Creek just before the mountain brow.

In areas of new development of Stoney Creek and Dundas, particularly above the mountain, wet ponds should be considered as a key potential element of a water quality control strategy, particularly to meet the IJC objective of reducing phosphorus inputs to the Great Lakes by 50 percent. In older areas of the Stoney Creek watershed and of Spencers Creek watershed below the mountain which are mainly developed, a retrofitting approach with wet ponds or equivalent technology is not addressed at this time. Rather it is suggested that an implementation plan for CSO control be developed and then this area be addressed in a decade when such issues as the need for wet ponds or better technology (i.e. phys-chem treatment becomes clearer).

8.3 Recommended Control Alternatives

The previous section outlined various control alternatives which, if implemented, would reduce the pollutant loading from the combined sewer system. In assessing the alternatives various factors including technical feasibility, effectiveness in reducing pollutant loading and operation and maintenance were considered.

This section summarizes the review of control alternatives and recommends specific alternatives for further evaluation. Summarized in Table 8.2 are the various options with qualitative rankings for criteria used in comparison of alternatives. These rankings (i.e. good, fair and poor) have the characteristics outlined below:

1. Good: -technically feasible and implementable;
   -high level of improvement and water quality benefit;
   -cost-effective;
   -high public and municipal acceptance;
   -low conflict potential; and
   -low environmental impact.

Table: 8·2 **COMPARISON AND RECOMMENDATION OF CONTROL OPTIONS**

Good ● Fair ◒ Poor ○ Future Study Required —

CONTROL OPTIONS	EVALUATION CRITERIA									
	Potential Level Of Improvement	Technical Feasibility	Cost Effectiveness	Implementation Considerations	Municipal+Public Acceptance	Conflict Potential	Environmental Assessment Process	Operation and Maintenance Considerations	Overall Rating	Recommended For Further Evaluation
SOURCE QUALITY CONTROL										
1. Street Sweeping	○	●	○	●	◒	●	●	○	○	NO
2. Public Education Programs	○	●	●	●	◒	●	●	●	◒	YES
3. Roof Downspout Disconnection	◒	◒	◒	◒	◒	●	●	●	◒	YES
4. Infrastructure Rehabilitation	○	●	○	◒	◒	●	●	○	○	NO
SEWER SYSTEM CONTROL										
5. Adjustment of Regulators	◒	●	●	●	●	●	●	●	●	YES
6. Swirl Concentrator	○	◒	◒	◒	◒	◒	◒	◒	◒	YES
7. Real Time Control	◒	●	◒	●	◒	●	●	●	●	YES
8. Offline Storage	●	●	●	●	●	◒	●	●	●	YES
9. Inline Storage	●	●	●	●	◒	●	◒	◒	●	YES
TREATMENT OPTIONS										
10. Improved Treatment at Woodward WPCP	●	—	●	●	●	●	●	●	●	YES
11. Plant expansion at Woodward WPCP	●	●	●	●	●	◒	◒	—	●	YES
12. Stormwater Management (Wet) Ponds	●	●	●	◒	◒	◒	●	◒	●	YES

2. Fair: –high level of improvement and water quality benefit;
–technically feasible with moderate constraints to implement;
–conflict potential;

–acceptance difficulties and multi–agent involvement; and
–straightforward environmental assessment process.

3. Poor: –low cost–effectiveness;
–low potential for implementation;
–minimal level of improvement and water quality benefit; and
–full environmental hearing required.

The alternatives have been divided into two general groups, Source Quality Control and Sewer System Control. An overall rating of good, fair or poor is given to each alternative. In general, alternatives which received a good rating were considered for detailed evaluation. Other alternatives which received a fair rating were assessed in qualitative fashion.

An overview for each evaluation criteria is given below:

Potential Level of Improvement

This criteria relates to the level of pollutant loading reduction that is achievable for each remedial alternative. For example, street sweeping or adjustment of flow regulators is limited with respect to the level of reduction that can be attained while storage facilities, if constructed to the appropriate size, can significantly reduce CSO frequencies and volumes.

Technical Feasibility

A remedial alternative is deemed to have a high level of technical feasibility if the technology has been proven to be effective elsewhere and the local conditions are such that the technology could be easily applied within the study area. In general, most of the alternatives were assessed a high rating with the exception of the swirl concentrator in which case the technology still has to be proven to be effective.

Cost-Effectiveness

Cost-effectiveness is a factor which directly relates the cost of the proposed alternative to the reduction in frequency or pollutant loading removal.

Implementation Considerations

The degree of difficulty to implement various remedial alternatives depends on various factors including funding, jurisdictional approval, site availability, policy revision, public/municipal acceptance and the regulatory process. Several of these factors are evaluated separately.

The ease of implementing is a key variable in defining the overall rating for a given alternative. A control alternative may be cost effective and result in a significant pollutant loading reduction. However, if the alternative cannot be implemented due to one or more of the factors described above, then the overall value of the alternative is extremely limited.

Municipal/Public Acceptance

Several alternatives may meet with political, municipal or public resistance. This may delay the process, or in the extreme result in public outcry. Public acceptance should be considered strongly in selecting a preferred alternative.

Several remedial alternatives may require additional operational funding, retraining of staff, or addition to staff. These factors are considered in assessing the rating for this criteria.

Conflict Potential

Several control alternatives may result in a potential conflict with existing or proposed usages of land or water. For example, harbour slips may be utilized for a Dunkers Flow Balancing System. However, if the slips are currently used for decking or loading of materials then a conflict exists. Underground tanks may also result in a conflict with existing or proposed land uses of a designated site.

Environmental Assessment Process

The Pollution Control Plan process is such that alternative remedial alternatives are evaluated in a systematic manner. Within Ontario, the requirements of the Environmental Assessment process also necessitates that the recommended strategy meet with approval from the regulatory agencies and the public. This involves, among other items, a comparison of the recommended strategy to other alternatives.

In developing a plan the proponent must, therefore, bear in mind that a certain alternative may not be easily approved or accepted by the public or regulatory agencies. This may result in a lengthy approval process or a full Environmental Hearing. Alternatives that may meet with resistance to approval are, therefore, given a low rating.

Operation/Maintenance Considerations

Several alternatives may require significant additional funding or staffing in order that the proposed measure continues to operate satisfactorily. Other alternatives, such as twinning an existing interceptor sewer may improve the existing conditions for operating and maintaining the system (i.e. flows could be directed to the new interceptor while the necessary operation/maintenance of the old sewer is carried out). In the former case a low rating would be applied while a high rating would be applied to the later case.

Overall Rating

As shown in Table 8.2, ten remedial alternatives were assigned a fair or good rating and are thus recommended for further evaluation. The results of the subsequent evaluation are provided in the next chapter.

9 – EVALUATION OF RECOMMENDED CONTROL ALTERNATIVES

9.1 General

Chapter 7 established the criteria for evaluating the remedial alternatives. As outlined in that section, two standards would be used, these being:

- alternatives to reduce CSO's to four overflows/ year; and
- alternatives to reduce CSO's to 1 overflow/year.

As outlined in Chapter 7, selecting these two standards would bracket the required level of pollutant loading reduction as outlined in the HHRAP documents.

Chapter 8 defined a preliminary list of remedial alternatives that could be evaluated as well as a refined list that warranted further evaluation. An overview as to the methodology to evaluate the preferred remedial alternatives is summarized in Chapter 7. Further detail is given below.

The evaluation of the sewer system may be carried using one of many approaches. For example, design events may be used as a basis or continuous modelling (e.g. 1 year, 2 years or 20 years may be used). The evaluation may include flows only or flows and concentrations (i.e. pollutant loading). One further consideration involves the complexity of the system to be modelled. In one extreme, virtually every sewer system element could be modelled. Conversely, the system may be significantly simplified to the point where all flows are assumed to go to either the WPCP or to the receiving body of water (i.e. overflow devices are ignored).

The approach used in this study involved:

- defining the system in reasonable detail with respect to the hydraulics of the system (i.e. approximately 250 sewer segments were included in the analysis) and with respect to the timestep used in the modelling (i.e. 10 minutes).

- defining the variability in CSO's volume to the receiving bodies of water for three extreme years (i.e. dry, typical and wet) in order to define the difference in loadings;

- evaluating numerous remedial alternatives based on a design event basis or using continuous modelling for a representative sewershed;

- ensuring that the results could be extrapolated, if necessary, to provide results for varying conditions.

In summary, this approach was selected as:

- it allows a reasonable number of remedial alternatives to be evaluated which are based on existing conditions and constraints. In other words, realistic alternatives which account for capacity constraints within the system or WPCP, or land availability, would be evaluated. Furthermore, reasonable preliminary sizing of the facilities could be established.

- further detailed evaluation as was discussed in Chapter 7 would have required considerable additional effort and would have resulted in a minimal increase in the reliability of the results;

- the available data by which the model could be calibrated/validated is limited to five monitoring sites. No water quality data were available for validation of computed results.

Table 8.2 summarized the findings from the screening process of the various remedial alternatives. In total, ten control alternatives are recommended for further evaluation. As outlined below, several of the alternatives will be evaluated in detail and in conjunction with other alternatives. One of the alternatives (e.g. roof downspout disconnection) will be evaluated for a typical sewershed while three of the alternatives (e.g. swirl concentrators, Real Time Control and public education) will be addressed only in a qualitative manner (see discussions, Section 8).

9.2 Evaluation of Primary Control Alternatives for CSO's

Based on the review of potential remedial works to reduce CSO's, the primary control alternative considered was the storage of CSO's in inline and/or offline storage facilities. This structural alternative is commonly used throughout North America and Europe and has been proven to provide high levels of CSO control. Five combinations of inline and/or offline CSO storage facilities were considered in order to determine the most effective balance between inline and/or offline storage.

Offline Storage Assumptions

Through discussions with Region staff it was concluded that the size of offline storage facilities would be determined assuming:

- circular storage tanks;and
- a storage depth of 10 m.

Minimum land requirements for the storage facilities were determined assuming a square parcel of land was to be purchased with a length/width equivalent to the tank diameter plus five metres. The 5 m 'buffer strip' around each facility was included to provide additional space for construction and maintenance. It is also assumed that pumping would be required to return stored flows to the interceptor.

The procedures outlined above were developed to provide a systematic way of estimating tank sizes and land requirements. Actual tank sizes, dimensions and land requirements will have to be determined for each proposed tank during detailed design.

Construction and land costs associated with the CSO storage facilities are reviewed in Appendix E.

Inline Storage Assumptions

Inline storage of CSO's is another option available to control CSO's in the Hamilton area. Inline storage can be obtained by constructing an over-sized interceptor sewer which can both store CSO's and convey CSO's to the WPCP for treatment. A review of the sewer system infrastructure in the area suggests that inline storage would be most feasible in the area west of the Woodward Avenue WPCP and east of Highway 403. Characteristics of this area that make inline storage an attractive option are:

- numerous CSO outfalls exist;
- land is not readily available for the construction of offline storage facilities.

An additional benefit of the inline storage facility is that should the existing western sanitary interceptor need to be serviced, a majority of the sanitary flows from the western interceptor could be re-routed and conveyed by the proposed inline tunnel to the WPCP for treatment.

Through meeting with Region staff and reviewing the combined sewer system in Hamilton's east end the proposed alignment of the inline facility is primarily along the spur railway line between Burlington Street and Barton Street. The spur line was selected since:

- it is located close to the existing flow regulator/CSO sites (i.e. piping costs would be minimized);
- an easement along the spur line currently exists;
- construction of the tunnel would not significantly affect traffic patterns; and
- the spur line was built for low speed, rather than high speed, trains (therefore the risk of disturbing train traffic is less).

Through a meeting between the study team and Region staff, it was also concluded that while an inline facility should terminate in the vicinity of the WPCP wet well, it should not flow directly into it. This decision was made to minimize the probability of wet-well pumphouse flooding and WPCP process bypassing since a large additional inflow to the WPCP could not be handled by either the wet well pumphouse or the WPCP processes. Rather, stored flows in the inline facility are to be pumped to the WPCP when the capacity exists at the plant to treat the stored sewage. An outfall from the inline facility to Hamilton Harbour or the Windemere Basin may also be required to control surcharging in the inline facility due to excess inflows.

The five alternatives are outlined in Table 9.1 and are shown graphically in Figures 9.1 through 9.5. A brief description of each alternative is given below. Detailed summaries of each alternative are included in Appendix D. Remedial works and associated costs for the Woodward Avenue WPCP are included in Section 9.4.

9.2.1 Alternative No. 1 - Separate Offline Storage Facilities at Each Outfall (Figure 9.1)

As the description implies storage facilities would be constructed at each outfall. In total there would be 22 facilities, in addition to the existing Greenhill Avenue facility. This alternative is somewhat limited in that land availability in the Gage Avenue to Dunn Street area and other isolated areas (e.g. Wellington) is limited. This would necessitate that existing houses be purchased and torn down to

TABLE 9.1

CSO STORAGE ALTERNATIVES AND ASSOCIATED COSTS

Option and Description	Number of Offline Facilities*	Length & Diameter of Inline Tunnel	4 CSO/Year Control Strategy				1 CSO/Year Control Strategy			
			Cons'n Costs		Land Costs	Total Costs **	Cons'n Costs		Land Costs	Total Costs **
			Tanks	Tunnel			Tanks	Tunnel		
1. Separate Offline Storage Facilities at each outfall	22	-	72.1	0	4.0	76.1	115.3	0	5.8	121.1
2. Offline Storage Facilities with some Lumped Facilities	18	-	70.5	0	3.7	74.2	113.8	0	5.2	119.0
3. Offline Storage Facilities and Inline Storage from the Woodward Avenue WPCP to Gage Avenue	12	Length = 4.5 km D=7.0m for 4 CSO's D=9.0m for 1 CSO	42.5	52.0	1.3	95.8	72.3	65.0	1.8	139.1
4. Offline Storage Facilities and Inline Storage from the Woodward Avenue WPCP to Wentworth Avenue	10	Length = 6.2 km D=7.0m for 4 CSO's D=8.5m for 1 CSO	34.2	71.0	0.9	106.1	60.0	86.8	0.8	147.6
5. Offline Storage Facilities and Inline Stroage from the Woodward Avenue WPCP to Highway 403	9	Length = 11.2 km D=6.0m for 4 CSO's D=7.0m for 1 CSO	24.0	112.0	0.3	136.3	44.7	129.0	0.9	174.6

* Excluding the existing Greenhill CSO Storage Facility.
All costs in Million Dollars. Cost estimate data is included in Appendix E.

** Excludes WPCP expansion costs of $90 and $100 million for the 4 and 1 CSO/year control strategies, respectively.

REGIONAL MUNICIPALITY OF HAMILTON - WENTWORTH

POLLUTION CONTROL PLAN

LEGEND

- Combined Sewershed
- Combined Sewersheds Draining To A Common CSO Storage Facility
- CSO Outfall
- Offline Storage Facility
- Lumped Offline Storage Facility
- Proposed CSO Tunnel Interceptor

CSO STORAGE OPTION #1
SEPARATE OFFLINE STORAGE FACILITIES AT EACH OUTFALL

accommodate the construction of the proposed facilities or easements within existing industrial lands be purchased.

In summary, the estimated costs are $76.1 million for 4 CSO's/year and $121.1 million for 1 CSO/year. The facilities range in size from 2,000 m^3 (Queenston Avenue, 4 CSO's/year) to 114,000 m^3 (additional Greenhill facility, 1 CSO/year). The total land requirements ranges between 7.7 ha (4 CSO/year) to 11.8 ha (1 CSO/year).

9.2.2 Alternative No. 2 – Offline Storage Facilities With Some Lumped Facilities (Figure 9.2)

This alternative is similar to Alternative No. 1 above, with the exception that flows from several overflow locations would be conveyed to a central location. The criteria used to combine two or more storage facilities are listed below.

1. Outfalls should be within 500 m of each other in order to minimize piping costs.

2. If possible, use favourable land to locate a facility. For example, land owned by the City or Region (e.g. parkland) would be favourable since it would be less expensive and easier to acquire than private lands.

Using these criteria the study team and Region staff identified three sites where CSO storage facilities could be centralized. The proposed sites are:

- public lands by Highway 403, Main Street and King Street where CSO's from the King and Dundurn outfalls could be combined;
- public lands by the Queen, Hess and Strachan (not modelled) outfalls; and
- Eastwood Park where CSO's from Catharine, Ferguson, Wellington, and possibly James, may be combined.

This would reduce the number of facilities to 18 (excluding the existing facility on Greenhill Avenue). Similar limitations to land availability as outlined in alternative No. 1 would exist.

The estimated costs for this storage option are $74.2 million and $119.0 million for the 4 and 1 CSO/year control strategies, respectively. These costs are slightly less than those estimated for CSO storage option #1 since lumping of CSO storage facilities results in cost savings due to economies of scale.

REGIONAL MUNICIPALITY OF HAMILTON - WENTWORTH

POLLUTION CONTROL PLAN

LEGEND

- Combined Sewershed
- Combined Sewersheds Draining To A Common CSO Storage Facility
- CSO Outfall
- Offline Storage Facility
- Lumped Offline Storage Facility
- Proposed CSO Tunnel Interceptor

CSO STORAGE OPTION #2
OFFLINE STORAGE FACILITIES WITH SOME LUMPED STORAGE FACILITIES

paul theil associates limited

9.2.3 Alternative No. 3 - Offline Storage Facilities and Inline Storage from the Woodward Avenue WPCP to Gage Avenue (Figure 9.3)

The following three alternatives look at a combination of inline and offline storage facilities. The alternatives were established based on discussions with the technical committee. The control alternatives centre around twinning the existing western interceptor sewer to provide additional inline storage. Flows up to the design event would then be intercepted by the proposed trunk sewer and conveyed to the Woodward WPCP. An additional benefit would be realized by the final three alternatives in that should the existing western interceptor need to be serviced a majority of the flows could be conveyed to the proposed trunk sewer.

Alternative No. 3 involves the construction of an inline storage facility from the Woodward WPCP to Gage Avenue. Based on existing inverts, utility constraints, etc., the proposed interceptor would be tunnelled. In total there would be 4.5 km of inline storage and 12 offline tanks. Preliminary discussions with municipal staff would suggest that open space areas could be utilized for constructing the offline facilities.

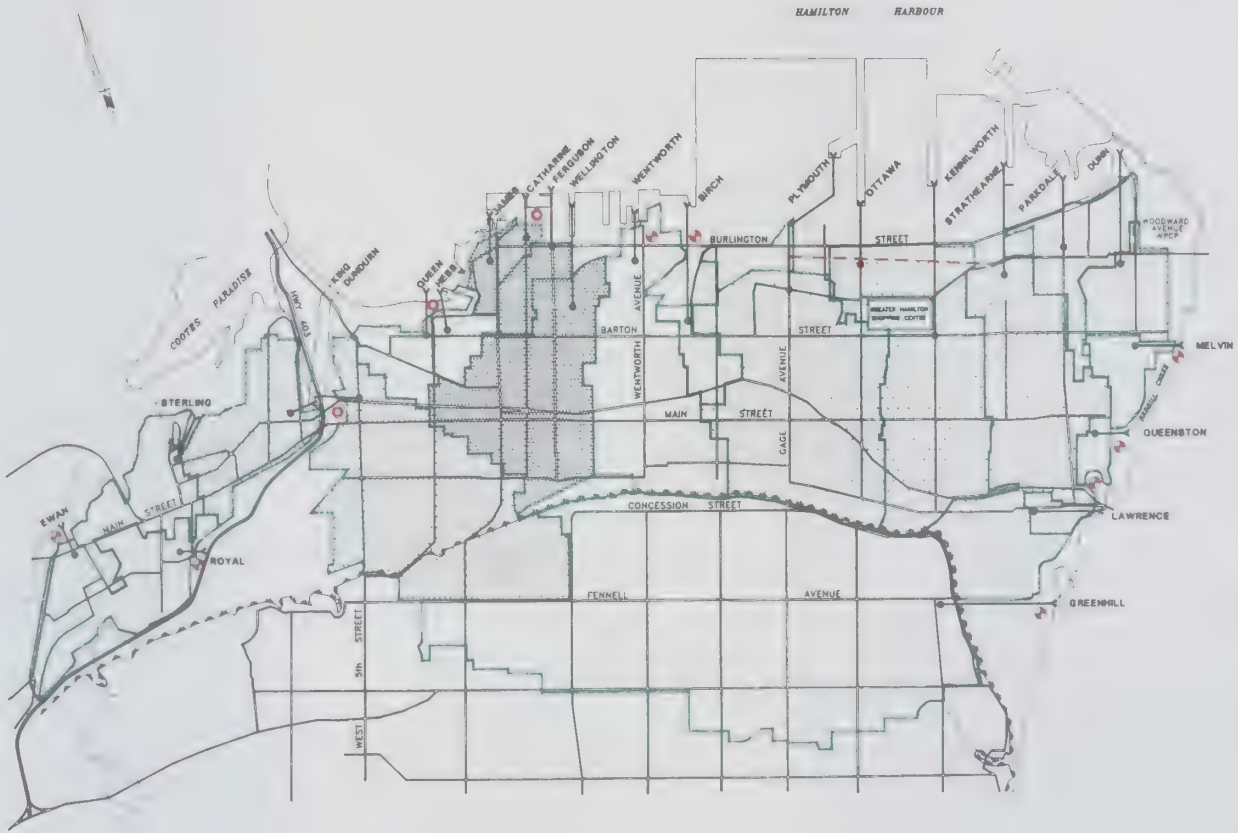
Table 3 in Appendix D provides a detailed breakdown of Option 3 and Table 9.1 summarizes the results. Overall, adopting a 4 CSO/year control strategy with this storage option will cost approximately $95.8 million. Approximately 45 percent of this cost is required for the construction of the offline storage facilities, while 55 percent of the cost is required for the inline tunnel.

The estimated cost to construct the proposed works to reduce CSO's to one event per year is $139.1 million of which $65 million would be required for the 9.0 m diameter tunnel and $74.1 million for the offline facilities.

9.2.4 Alternative No. 4 - Offline Storage Facilities and Inline Storage from the Woodward Avenue WPCP to Wentworth Avenue (Figure 9.4)

This alternative is similar to alternative No. 3, with the exception that the proposed inline storage facility has been extended westerly to Wentworth Avenue. In total the length of the proposed inline storage facility is 6.2 km. A total of 10 offline storage facilities would also be required to contain CSO's.

Adopting Option 4 with a 4 or 1 CSO/year control strategy would cost approximately $106.1 and $147.6 million, respectively.



REGIONAL MUNICIPALITY OF HAMILTON - WENTWORTH

POLLUTION CONTROL PLAN

LEGEND

- Combined Sewershed
- Combined Sewersheds Draining To A Common CSO Storage Facility
- CSO Outfall
- Offline Storage Facility
- Lumped Offline Storage Facility
- Proposed CSO Tunnel Interceptor

CSO STORAGE OPTION #3 OFFLINE STORAGE FACILITIES AND INLINE STORAGE FROM THE WOODWARD AVENUE WPCP TO GAGE AVENUE

REGIONAL MUNICIPALITY OF HAMILTON - WENTWORTH

POLLUTION CONTROL PLAN

LEGEND

- Combined Sewershed
- Combined Sewersheds Draining To A Common CSO Storage Facility
- CSO Outfall
- Offline Storage Facility
- Lumped Offline Storage Facility
- Proposed CSO Tunnel Interceptor

CSO STORAGE OPTION #4 OFFLINE STORAGE FACILITES AND INLINE STORAGE FROM THE WOODWARD AVENUE WPCP TO WENTWORTH AVENUE

9.2.5 Alternative No. 5 – Offline Storage Facilities and Inline Storage from the Woodward Avenue WPCP to Highway 403 (Figure 9.5)

This alternative is similar to the previous two alternatives. For this alternative inline storage would be extended westerly to Highway 403 (11.2 km in total). The number of offline facilities would be reduced to 9.

Option 5 is the most expensive CSO storage plan proposed. Costs range between $136.3 million and $174.6 million for the 4 and 1 CSO/year control strategies, respectively. Costs are excessive for this option primarily because tunnelling is expensive and the length of the inline tunnel is approximately double that proposed for Option #4.

Assessment of Alternatives No. 1 to No. 5

A review of the five primary control alternatives by the study team and technical committee suggested that either Option 3 or Option 4 would be most feasible for the reasons outlined below.

1. Land availability is limited in Hamilton's east end as it is highly industrial, making Options 1 and 2 difficult to implement.

2. Capital costs associated with Option 5 are excessive.

3. Options 3 and 4 have an additional benefit in that should the existing western interceptor need to be serviced, a majority of the flows could be conveyed to the WPCP through the proposed trunk sewer.

Also, prior to the detailed design of any offline CSO storage facility suggested herein, the study team recommends that the proposed volume of a facility be confirmed. This recommendation has been made because this study necessitated simplifying the Region's extensive combined sewer system to simple lumped catchments. Therefore, although every effort was made to correctly define each CSO outfall's drainage area, some simplifications and assumptions had to be made, especially in the City's older areas (i.e. the Catharine, Ferguson and Wellington area) where combined sewer systems are interconnected. CSO storage volume confirmation would, therefore, be advisable, and may be obtained through flow monitoring in the existing CSO outfalls/trunk sewers, and through additional field surveys.

REGIONAL MUNICIPALITY OF HAMILTON - WENTWORTH

POLLUTION CONTROL PLAN

LEGEND

- Combined Sewershed
- Combined Sewersheds Draining To A Common CSO Storage Facility
- CSO Outfall
- Offline Storage Facility
- Lumped Offline Storage Facility
- Proposed CSO Tunnel Interceptor

CSO STORAGE OPTION #5
OFFLINE STORAGE FACILITIES AND INLINE STORAGE FROM THE WOODWARD AVENUE WPCP TO HIGHWAY 403

Additional analysis of the proposed lumped (inline and offline) CSO storage facilities may also be warranted prior to detailed design and construction. One aspect that could be investigated can be outlined by the following question. Is it reasonable to assume, as was done in this study, that the 25 mm and 35 mm design storms are evenly distributed over the entire area serviced by a lumped facility? Of immediate interest is the inline tunnel facility which has been proposed to service an area between 1,600 and 2,000 ha (Options 3 and 4). If the average thunderstorm cell covers an area less than this, then less rainfall may actually be deposited on the "serviced" area under typical conditions. If this were the case, then less CSO volume would have to be stored, and capital costs for inline tunnel storage could be decreased. Further analysis of the lumped CSO storage facilities may, therefore, be warranted.

9.3 Evaluation of Secondary Remedial Alternatives for CSO's

Table 8.2 summarized the remedial alternatives which should be considered further for reducing CSO's to the receiving body of water. Section 9.2 discussed the alternatives which will have a major impact on CSO's. The section deals with several alternatives, which are beneficial in reducing CSO frequencies or pollutant loadings but which individually, will not significantly reduce the pollutant loading to the receiving bodies of water.

The remedial alternatives which are considered in this section include:

- adjustment of overflow regulators; and
- disconnection of roof downspouts;
- swirl concentrators with tanks;
- public education;
- Real Time Control.

9.3.1 Adjustment of Overflow Regulators

Figure 4.1. illustrates the variation in the threshold capacities for each of the 28 flow regulators. In summary, the settings at several of the flow regulators are such that a minimal amount of flows above the dry weather volumes may be directed to the interceptor prior to an overflow resulting (the threshold value in this case is approximately 3 l/s/impervious ha). Conversely, five major overflows have threshold levels which exceed 25 l/s/impervious ha. In this case a considerable amount of flow

beyond the dry weather component would be diverted to the interceptor sewer prior to an overflow resulting.

The objective of adjusting the flow regulators would be to reduce the frequency and volume of overflows in areas which are more environmentally sensitive (ie: Cootes Paradise) or where existing (or proposed) recreational uses are undertaken (i.e. the LAC property). This alternative would be of benefit in the short to medium term (5–20 years) until the major control alternative is in place.

Three factors have to be considered when evaluating this alternative. These include:

- the impact within the sewer system due to the diversion of additional flows to the interceptor;
- the impact at the Woodward Avenue WPCP;
- the ease with which the overflow regulator may be adjusted.

The last point above is important since if the adjustment of the overflow regulators is to be an effective interim solution then it should be both inexpensive and easy to apply. For example, opening a sluice gate or increasing the height of an overflow weir would be inexpensive and an effective means of directing more sewage to the WPCP, provided additional capacity exists in the pipe conveying flows from the regulator to the sanitary interceptor. Conversely, if no additional capacity exists in the sanitary "feeder" pipe, then the only means of directing more sewage to the sanitary interceptor would be to replace the "feeder" pipe. Pipe replacement would be substantially more expensive than regulator adjustment.

Appendix B provides a detailed information on the sites where flow regulator adjustments may be made.

Minor Flow Regulator Adjustment to Reduce CSO's

Outfalls where CSO frequencies and volumes may be reduced by simple adjustment of the flow regulators are listed in Table 9.2. This list is much shorter than the total number of flow regulators since field investigations indicated the majority of the flow regulators fully utilize all available capacity of the sanitary feeder pipes. The estimated increase in flows to the sanitary interceptor provided in Table 9.2 are based on hydraulic characteristics of the sites obtained from detailed drawings and

TABLE 9.2

CSO OUTFALLS WHERE CSO FREQUENCIES AND

VOLUMES MAY BE REDUCED BY FLOW REGULATOR ADJUSTMENTS

Outfall	Flow Regulator* Requiring Adjustment	Potential Adjustment(s)	Estimated Increase (m^3/s) in Inflows to Sanitary Interceptor	
			From	To
King	Glen/Tope (4810)	increase weir height	1.00	1.25
Dundurn	Chedoke/403 (4800)	increase weir height	1.40	2.80
Hess	Stuart/McNabb (1240)	open sluice gate and increase weir height	0.08	0.16
James	James/Guise (1220)	open sluice gate and increase weir height	0.08	0.16
Catharine	Catharine/Brock (5115)	open sluice gate and increase weir height	0.08	0.16
Wentworth	Rosemary/ Wentworth (2165)	increase weir height	0.52	0.60

* These regulators divert combined sewage to either the WPCP or CSO outfalls

TABLE 9.3

TYPICAL YEAR (1989) CSO CHARACTERISTICS

BEFORE AND AFTER FLOW REGULATOR ADJUSTMENT

Outfall	Existing Conditions		After Regulator Adjustment	
	Frequency	Volume	Frequency	Volume
King	20	72,600	13	59,000
Dundurn	29	529,200	23	215,000
Hess	13	13,200	13	12,500
James	24	16,500	15	11,900
Catharine	27	16,500	17	10,500
Wentworth	31	427,000	31	394,000

through field investigations. Generally, the increased flows assume that the maximum possible flow is conveyed from the flow regulator to the sanitary interceptor before CSO's commence. The influence of these adjustments on CSO frequencies and volumes were ascertained for the typical year (1989). Table 9.3 summarizes the results and indicates:

- CSO frequency decrease from zero to approximately 40 percent
- CSO volumes decrease from approximately 5 to 60 percent during a typical year.

Flow regulator adjustment can, therefore, be a feasible interim strategy for reducing CSO's.

With the exception of the increased flows from the Glen/Tope and King/Dundurn flow regulators to the sanitary interceptor, the increase in WPCP flows is not expected to be damaging to WPCP processes/pumphouse as the additional total inflow is equivalent to 6 MGD, or less than a 10 percent increase in WPCP inflows over dry weather flow conditions. Should the additional inflows from the Glen/Tope and King/Dundurn flow regulators become excessive, WPCP staff have the option to activate the on-site WPCP controlled sluice gates to divert all the flow upstream of these regulators to Chedoke Creek.

The influence of flow regulator adjustment on sewer surcharging/basement flooding was assessed qualitatively since the TRANSPORT block of SWMM 4 does not model surcharged sewer conditions. Nonetheless, it is expected that while western interceptor sewer surcharging may increase, basement flooding should not occur since the sanitary interceptor is deep (i.e. usually 10 to 20 m below grade) and hence is below basement elevations, (i.e. usually 2 to 5 m below grade) and appears to have few direct service connections to it. A segment of the western interceptor where sewer surcharging, and its effects, may be an issue is along the line that services the James and Catharine catchments (Figure 88143-B-1). This line is shallower (i.e. 6 to 10 metres) than most sections of the western interceptor and may have more direct service connections.

Major Flow Regulator Adjustment to Reduce CSO's

Two locations were identified where major "adjustments" to the flow regulators may significantly reduce CSO frequencies and volumes - at the James/Guise flow regulator (James outfall) and at the Catharine/Brock flow regulator (Catharine outfall). These regulators control CSO's in the James and Catharine subcatchments and both outfalls outlet to potential human contact areas (e.g. marina).

Servicing these subcatchments is a 1500 mm (60") sanitary interceptor which originates near the James/Guise flow regulator and travels along James/Burlington/Mary to the main line of the western interceptor at Ferrie Avenue. The branch of the sanitary interceptor serves a mere 48 ha and is therefore, grossly oversized. The sewer is large since tunnelling was used to construct the line. The potential exists, therefore, to route more flows from the James/Catherine combined trunk sewers, through the flow regulators, to the sanitary interceptor. This could be accomplished by upgrading the sanitary feeder pipes between the flow regulators and the interceptor. A review (Table 9.4) of the hydraulic characteristics of the system in this area suggests the potential exists to route approximately 50 percent of all combined sewage in the area directly to the sanitary interceptor. Currently, the flow regulators use less than 20 percent of the sanitary interceptor capacity.

TABLE 9.4
PIPE FLOW CAPACITIES IN THE
JAMES AND CATHARINE AREA

Pipe	Estimated Capacity (m^3/s)
James Street Combined Sewer 4' x 4' @ 1.25 %	2.3
Catharine Street Combined Sewer 42" @ 0.5 %	2.0
Sanitary Interceptor on James/Burlington/Mary 60" @ 0.10 %	2.0

Another remedial option available in this area is to route only flows from the James Street combined trunk sewer to the interceptor, while storing CSO's from the Catharine/Brock regulator in the proposed lumped storage facility in Eastwood Park (discussed in Section 9.2.2). This may be a more effective course of action for the reasons outlined below.

1. Routing CSO's from the James Street outfall to the proposed Eastwood Park CSO storage facility would be expensive, as approximately 450 m of pipe would have to be installed under existing roads. An estimated cost to install this piping is $450,000.00

2. It should be relatively inexpensive to route CSO's to the sanitary interceptor from the James/Guise flow regulator since less than 10 m of pipe is required.

3. The existing sanitary interceptor should be able to accept all flows from the James Street combined sewer, rather than approximately 50 percent of the flows from both Catharine and James trunk sewers. Consequently, additional remedial works (e.g. an off-line CSO tank) near the James Street outfall may not be required to control CSO's.

In order to limit inflows to the sanitary interceptor/WPCP it may be necessary to install a WPCP-controlled sluice gate at the James/Guise flow regulator to divert flows away from the sanitary system during severe rainfall events. A detailed study of this portion of the Region's infrastructure could provide more insight to the possibilities available to control CSO's at the James outfall.

9.3.2 Disconnection of Roof Downspouts

An assessment of the effect of roof downspout disconnection on CSO frequencies and volumes was conducted for the representative Strathearne outfall sewershed. Using the results of the City of York's program it was estimated, on average, that roof downspout disconnection would decrease percent impervious values by approximately 5 percent. A 5 to 10 percent reduction would also occur in the impervious areas with no depression storage contributing to runoff. The affect of these changes on CSO volumes are summarized in Table 9.5. Overall, CSO volumes decrease by approximately 5 percent as a result of roof downspout disconnections.

TABLE 9.5
INFLUENCE OF ROOF DOWNSPOUT DISCONNECTION
ON CSO VOLUMES FOR THE STRATHEARNE OUTFALL

Conditions	CSO Volume (m^3) For		Percent Decrease In CSO Volume
	Existing Conditions	Roof Downspout Disconnection	
Typical (1989) Year	309,000	284,000	4
35 mm Design Storm (1 CSO/Year Control Strategy)	55,500	50,000	5
25 mm Design Storm (4 CSO/Year Control Strategy)	36,500	32,800	5

It can be expected, therefore, that the capital costs to store CSO's would also reduce by approximately 5 percent due to roof downspout disconnection, since costs vary proportionally with volumes. This

would result in approximately a 5 million dollar savings, as the capital cost for CSO storage is expected to be approximately 100 million dollars (Section 8). Therefore, the costs required to execute a roof downspout disconnection program are approximately equivalent to any anticipated savings in the capital costs for CSO storage works.

As such, a program of this type should initially be restricted to areas where additional problems may exist (i.e. localized basement flooding).

9.4 Evaluation of Woodward Avenue WPCP Alternatives

Based upon the existing operating policy of the WPCP and the additional flows that will be diverted to it, the following order of priority for handling additional flows was adopted.

i) Treatment of existing flows coming to the plant. This involves
   - Treatment of existing dry weather flows
   - Improved treatment of existing stormwater flows
   - Treatment of flows presently bypassed around the plant

ii) Treatment of Addition Storm Flows
   - Treatment of Flows caused by redirecting overflow regulation
   - Treatment of additional CSO volumes derived from storage as storage units are built.
   - Treatment of all stormflows for the Region of Hamilton–Wentworth for a specified design storm.

To achieve these objectives and to enhance other treatment objectives (nitrification), the major WPCP alternatives are:

1. Modifications of Present Plant
   a) Optimization of the Present Plant
   b) North Side Settler Modification and Use of Step Feed

2. Plant Expansion

The simulations used to investigate these alternatives are documented in BEAK, (1991). Pertinent results are summarized in Appendix G.

Based upon these alternatives, the expansion requirements were evaluated, the land available examined for plant expansion, and cost estimates provided.

9.4.1 Interim Alternatives for Woodward Avenue WPCP

A variety of interim alternatives were evaluated using the process simulation model. These alternatives are given in Appendix G and were mainly designed to maximize treatment plant objectives (BOD, suspended solids and TKN removal) on an average daily basis while improving the capabilities of the present plant to handle storm related flows.

Based upon this analysis, the major modifications to the present plant which could improve the ability of the plant to handle storm related flows are:

1. Optimization of Present Plant
   - operate the sludge resistance time (SRT) at approximately 8 days on the north side and 7 days on the south side to achieve nitrification to keep the MLSS concentration as low as possible to minimize washout on the north side.

2. Present Plant Modification
   - install hydraulic structures and measurement devices to control the flow split between the north and south side for short periods of time
   - construct internal modifications to the north side settlers (eg., baffles, flow distribution control, etc.) to achieve better performance of the north side settlers
   - institute step feed operating policies during storm-flows
   - suitably modify operation of the present aerators to provide more oxygen transfer into the aeration tanks particularly during step feed operation

These measures appear capable of handling the different sizes of flows given in Table 9.6 without bypassing the secondary treatment reactors and achieving a performance for average daily suspended solids in the plant effluent of 25 mg/L and achieving significant nitrification. The simulations indicate that the largest effect on plant performance under storm conditions should be achieved by improving the final clarifiers on the north side of the plant.

TABLE 9.6

RERATING OF EXISTING PLANT BASED UPON PLANT IMPROVEMENTS

Plant Modification	Instantaneous Peak Flow*	Average Daily Flow*
1. No Modification	--	Present Operating Policy
2. Step Feed North Side	490 MLD	350 MLD
3. Improved Final Clarifier North Side	620 MLD	450 MLD
4. Step Feed; Improved Settlers North Side	630 MLD	460 MLD

*Approximate Values

Accordingly, this option would be able to handle the following flows:

- existing dry weather flows;
- improved treatment of existing stormwater flows;
- treatment of flows presently bypassed around the plant.

Several studies need to be completed, however, before retrofitting the plant to improve the final clarifiers and for step feed needs to be carried out after several studies have been completed. These studies, some of which are ongoing, include:

1. Studies to define the cause of poor performance of the north side secondary clarifiers, since opinions on the cause varies from professional to professional. One necessary study, dye tracer studies, are currently scheduled for the near future.

2. A study to define the most appropriate methods to improve controlling flow distribution between the north side and south side during storm flows.

3. A study to define the performance improvements possible in the existing mechanical aerations; this study has been initiated.

4. A full scale demonstration of step-feed of the plant and subsequent re-evaluation of the model calibration used in this study to evaluate the plant. The full scale demonstration is planned in the immediate future.

When these ongoing studies are complete, a data base will be available which will define the best options for retrofitting the plant and for estimating the associated costs.

9.4.2 Plant Expansion Options for the Woodward Avenue WPCP

A variety of plant expansion options were evaluated using simulation studies (see Appendix G) such as:

- expansion of only the secondary clarifiers;
- expansion of both the primary clarifiers and secondary clarifier; and
- expansion of the primary clarifiers, secondary clarifiers and aeration tanks.

In evaluating the expansion option, attention was given to plant expansions required to handle the storm volumes not stored plus the CSO storage for both 4 overflows/season and 1 overflow/season with the ultimate population.

Both step–feed and non–step feed policies were evaluated. Because the simulations suggest that there is not a substantive difference in plant performance between step–feed and non–steep policies over a three to four day period of steady flows, the non–step feed option was used to size plant expansion. The degree of expansion finally selected is outlined below.

Sizing Options for 1 Overflow/Year Control

The following plant expansion was considered for purposes of treating volumes and CSO volumes stored in the proposed facilities (Storage Facility Options 3 and 4) located within the Municipal combined sewer system.

1. Add 2 pumps to the pumphouse to bring it to its full complement.

2. Expand the primary clarifiers by 2.4 times (i.e. add primary clarifiers equivalent to 1.4 times the existing primary clarifiers. This provides an allowance for a tank out of service).

3. Expand the secondary clarifiers by 1.6 times (i.e., add new secondary clarifiers equivalent to 0.6 times the existing secondary clarifiers. This provides allowance for clarifiers out of service based upon historical practice).

4. Expand the secondary aeration tanks by doubling the south side (i.e., provide additional aeration tanks equivalent in size to the present south size aeration tanks).

5. Add a new detritor unit in parallel to the existing detritor unit; connect this unit to the existing pump house in parallel with the existing detritor unit and then connect it to the new primary tanks.

6. Provide an equalization tank, in conjunction with Storage Options 3 or 4 at the WPCP, equivalent to the volume of stormwater directed to the plant during the day of the storm and not stored in the CSO facilities (160 ML for a one overflow event). The equalization tank is provided to control the storm surge as it arrives at the plant.

This option is estimated to cost approximately $100 million in capital costs.

Sizing Option for 4 Overflow/Year Control

Expansion requirement for this option are the same as given for the 1 Overflow/year control for the pumps (item 1 above), the secondary aeration tanks (item 4 above) and the detritor unit (item 5 above). Volume requirements for the other items are slightly smaller at.

1. Expand the primary clarifiers by 2.2 times
2. Expand the secondary clarifiers by 1.5 times
3. Provide equalization storage of 135 mL.

This option is estimated to cost $90 million in capital costs.

9.4.3 Explanation of Sizes Selected

The sizing can be explained as follows:

1. The primary and secondary clarifiers were sized based upon the design values of the existing reactors and the average flow rate to the plant during the release of stored CSO volumes.

2. The aeration tanks were sized based upon operation considerations.

3. The degree of expansion was checked with available land.

Detailed Description of Expansion of Primary and Secondary Clarifiers

The existing primary clarifiers are designed for 270 MLD (60 MIGD); the secondary clarifiers are designed for 405 MLD (90 MIGD). The secondary clarifiers do not function at this design level due to:

- poor performance of the north side settlers during storm flows;
- inability of the sludge return pumps on the south side clarifiers to handle a solids loading beyond a threshold value. In this expansion analyses, it was assumed that these problems would be resolved during the interim modification.

The necessary expansion of the primary and secondary clarifiers are as follow:

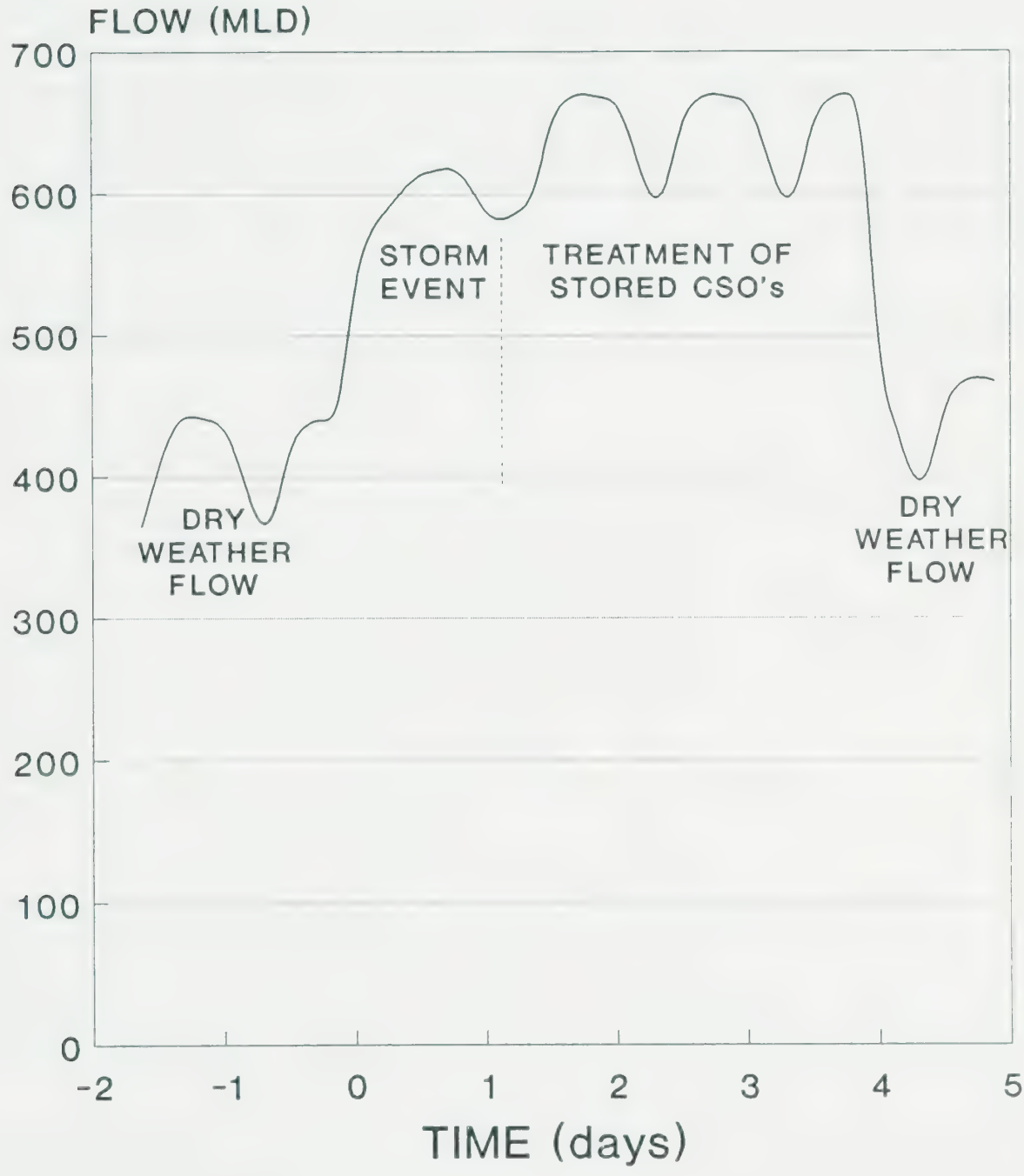
Design Evaluation	Flow Rate to Treatment Plant	Primary Clarifier Size	Secondary Clarifier Size
Present Size		270 MLD	405 MLD
4 Overflows/season	590 MLD	2.2 times	1.5 times
1 Overflow/season	650 MLD	2.4 times	1.6 times

Detailed Description of Expansion of Aeration Tanks

Expansion of the aeration tanks was based upon variations in the inflow hydrograph before, during and after treatment of stored CSO volumes. The influent flow hydrograph to the plant for treating a 1 CSO overflow/season event is sketched in Figure 9.6. It is basically composed of the following:

- 405 MLD during normal dry weather flow in the year 2025 (shown as day $^{-1}$);
- the flow would increase to 580 MLD during the first day, with some degree of peaking occurring due to stormflows, if the equalization tank is not built (shown as day 0 to 1);

FIGURE 9.6: INFLUENT HYDROGRAPH FOR TREATING STORMWATER & DRY WEATHER FLOWS FOR 1 CSO/YEAR CONTROL STRATEGY



- the flow would be approximately 650 MLD over the next three days as stored CSO volumes are released from storage at a constant rate (shown as day 1 to 4);
- the flow could return to 405 MLD (shown as day 5).

In addition, diurnal variations for each of these days are shown in the schematics:

Operating the plant during this hydrograph would pose tremendous operating difficulties for the plant staff, if substantial aeration tank volume were provided to handle an average flow rate of 650 MLD for a 3 day period because this complete volume of aeration tanks would never be required again for a long period of time. In addition, sludge age control would be difficult if the aeration tanks were fully expanded. Accordingly, the control option selected for the plant involves expansion of primary and secondary clarifier sufficient to handle the peak flows from storms and expanding the aeration tanks sufficient to handle dry weather flows on a sustained dry weather basis in 2025.

Land Availability

The review of the past layout of the plant and subsequently expansions indicate that there was sufficient land available to approximately double the complete plant. However, other modifications have been made to the plant to interfere with this general concept. Accordingly trial layouts were made with the above described expansion scheme; with some judicious layouts, it was established that there was sufficient land available for the above discussed plant expansion and also leave area for final effluent sand filters if required.

9.4.4 Woodward Avenue WPCP Planning

There are several additional studies underway at the Woodward Avenue plant (settling tank diffusion studies, step feed demonstration; measurement of aeration capacity) at the Woodward Avenue plant. There are additional effluent objectives which may require installation of sand filters for final effluent polishing; filters would substantially reduce the size of final clarifiers required. Addition data has been recently gathered which would provide additional calibration of the model used in this study.

Based upon these considerations, it is suggested that the following steps be carried out over the next 5 year time frame to develop an improved WPCP Expansion Plan to that developed in this study.

1. Evaluation plant improvements possible from the ongoing studies (1991).

2. Based upon additional calibration data, and refined objectives for plant performance, examine the simulation model calibration and adjust plant expansion requirements into several time frames, for example:

   0–5 yrs.

   5–10 yrs.

   10–15 yrs.

   15–20 yrs.

   20–25 yrs.

   Any expansion should also take into account the peaking of the influent hydrograph until system storage units or equalization tanks are built (this could be done in 1992–93)
3. Initiate detailed studies of other treatment options (1992).
4. Develop a plan for plant improvements to the present plant.
5. Develop a detailed plan for plant expansion. We see this to involve the following elements.
   - adding additional primary clarifiers fairly immediately, since the present clarifiers are already overload
   - evaluate the performance of additional clarifiers and determine whether a biological treatment strategy for stormflows is the preferred option, or whether physical–chemical options should be used
   - determine whether additional aeration tanks and final clarifiers should be built, and if so, their size

9.5 Evaluation of Remedial Works for Stormwater

The recommended remedial work to reduce constituent loadings from separated storm sewer areas is to use stormwater detention (wet) ponds, or equivalent technology, in new developments.

No wet ponds currently exist in the Region, nor are there any policies in place requiring that wet ponds be installed in new developments. The objective of the analysis was, therefore, to illustrate the potential reductions in constituent loadings with wet ponds.

Table 9.7 compares the constituent loadings from separated storm sewer areas under:

- existing conditions;
- future development conditions; and
- future conditions with wet ponds installed in new developments only.

TABLE 9.7

SEPARATED STORM SEWER LOADINGS

TO HAMILTON HARBOUR/COOTES PARADISE

(MAY 1 TO OCTOBER 31)

Parameter	Percent Removal With Remedial Works *	Daily Loading (kg/day) for Specified Conditions		
		Existing Conditions	Future Conditions No Remedial Works	Future Condition With Remedial Works
SS	80	6,000	8,200	5,600
TS	80 for SS 0 for DS	27,000	37,000	34,000
TP	50	20	27	23
NH_3	20	21	25	23
TKN	50	86	111	89
Zn	40	9	13	11
FC (total counts)	20	$1.4x10^{13}$	$2.1x10^{13}$	$1.9x10^{13}$
BOD_5	10	500	740	700
Pb	70	7.7	11	8.2
Cu	40	1.9	2.8	2.3

* Remedial works correspond to stormwater management (wet) ponds or equivalent technology. Remedial works are assumed to be installed in future developments only, no remedial works are proposed for existing developments. Therefore, overall percent reduction in constituent loadings will be less than those listed.

Overall, the analysis indicates loadings increase by approximately 40 percent if no wet ponds are installed. However, if wet ponds are installed in the proposed development areas:

- SS loadings will reduce to slightly below existing values;
- TP, NH_3, TKN and Pb loadings will increase by only 10 percent;
- the remaining constituents will increase by approximately 20 to 35 percent.

Loadings could actually decrease below existing levels depending on the removal efficiency to be applied to runoff from new developments. For instance, implementation of wet ponds as rural areas are being developed and reduce the SS concentrations from an average of 150 mg/l to about 20 – 30 mg/l, compared to an average 100 mg/l for current urbanized areas. Overall, the use of wet ponds is expected to limit the increase in constituent loadings to approximately 10 to 25 percent, rather than the 40 percent increase expected if wet ponds are not installed in new developments.

Options also exist for controlling existing urban stormwater discharge. For example, existing stormwater detention (dry) ponds could be converted to wet ponds by either excavating an additional, permanent, storage volume below the invert of the outlet, and/or raising the elevation of the outfall pipe. Alternately, in older areas where neither wet nor dry ponds exist, wet ponds could be constructed within the downstream channels to treat the stormwater discharges within the stream and/or its tributaries. This alternative, for example, may be feasible in certain areas of Chedoke Creek. Concrete channels could also be naturalized, and given a pervious low flow channel along with vegetated banks. This would promote infiltration within the channel and provide for some nutrient uptake through the plants. These options, however, will not be evaluated quantitatively.

9.6 Reduction in Constituent Loadings With Proposed Works

Previous sections have addressed existing loadings to Hamilton Harbour/Cootes Paradise and to what extent loadings are expected to increase under future development conditions. Interim remedial works to reduce CSO's and improve Woodward Avenue WPCP performance have also been presented. The purpose of this section is to ascertain the effects of the recommended CSO, WPCP and stormwater remedial works on loadings. The recommended remedial works include:

- CSO control strategies of 1 and 4 CSO events/year;
- CSO storage alternatives 3 or 4;

- the proposed WPCP works to maintain current effluent quality; and
- stormwater management (wet) ponds in all future developments.

The analysis was conducted for a typical year (1989) under both existing and future conditions. Existing conditions assume:

- existing developments are as shown in Figure 2.3; and
- an average WPCP inflow of 320 MLD.

Future conditions assume:

- additional residential and commercial/industrial development as shown in Figure 2.3; and
- an average WPCP inflow of 428 MLD (i.e. a population of 500,000 served by WPCP).

For comparison purposes, all loadings were measured according to their percent increase or decrease relative to existing conditions. General results are tabulated in Tables 9.8 and 9.9 and Figure 9.7 illustrates the results for SS, TP and TKN. Detailed results are included in Appendix H. Conclusions which may be drawn from the analysis are outlined below.

1. CSO loadings reduced by 95 to 87 percent with the 1 and 4 CSO/year control strategies, respectively. CSO loadings are not expected to increase under future conditions since the combined sewer drainage area is essentially fully developed.

2. Under existing conditions, WPCP inflow volumes increase by approximately 6 percent under either CSO control strategy since more sewage is directed to the WPCP by the CSO storage facilities. Accordingly, since the WPCP remedial works are designed to maintain, rather than enhance effluent quality, WPCP loadings also increase (by approximately 3 percent). WPCP loadings do not increase to the same extent as WPCP inflow volumes since the additional sewage is more dilute than normal WPCP inflows.

3. Under future conditions, loadings from the WPCP are expected to increase by approximately 33 percent. Approximately 90 percent of this increase may be attributed to the expected increase in population in the sewershed. The remaining increase originates from the diversion of CSO flows to the WPCP by the CSO storage facilities.

TABLE 9.8

RELATIVE LOADINGS FOR DIFFERENT CONDITIONS

Conditions and Loading Type	Average Percent Increase (+) or Decrease (−) in Loadings Relative to Existing Conditions	Comments
Existing Conditions		
CSO Loadings for:		
– 4 CSO/year Control Strategy	−87	CSO loading decreases due to large CSO volume reduction
– 1 CSO/year Control Strategy	−95	
WPCP Loadings for:		
– 4 CSO/year Control Strategy	+3	WPCP loadings increase since CSO storage facilities direct more flow to the WPCP
– 1 CSO/year Control Strategy	+3	
Stormwater Loadings	0	No stormwater remedial works proposed for existing developments
Total CSO, WPCP and Stormwater Loadings for:		
– 4 CSO/year Control Strategy	See Table 9.9	None
– 1 CSO/year Control Strategy	See Table 9.9	
Future Conditions		
CSO Loadings for:		CSO loadings same as existing conditions since combined sewer area will essentially remain unchanged
– 4 CSO/year Control Strategy	−87	
– 1 CSO/year Control Strategy	−95	
WPCP Loadings for:		30 percent increase in WPCP loadings due to population increase
– 4 CSO/year Control Strategy	+33	3 percent increase in WPCP loadings due to CSO diversion
– 1 CSO/year Control Strategy	+33	Loadings increase with volumes since WPCP effluent quality is not improved under current plan
Stormwater Loadings		
– with no remedial works	+39	Separated sewer area increases by 66%
– with remedial works	+17	Wet ponds decrease loadings
Total CSO, WPCP and Stormwater Loadings		
– with no remedial works	See Table 9.9	None
– with remedial works (average)	See Table 9.9	None

TABLE 9.9

CONSTITUENT LOADINGS TO HAMILTON HARBOUR

Constituent	Loading For Existing Conditons (kg/day)	Percent of Existing Loading Contributed by CSO's	Percent Increase (+) or Decrease (-) in Total CSO, WPCP and Stormwater Loadings for Stated Conditions				
			Existing Level of Dev. With Recommended CSO and WPCP Remedial Works		Future Conditions With No Remedial Works*	Future Conditions With Recommended CSO, WPCP & Stormwater Remedial Works	
			4 CSO/Yr Control Strategy	1 CSO/Yr Control Strategy		4 CSO/Yr Control Strategy	1 CSO/Yr Control Strategy
SS	15,700	17	-14	-15	+27	-3	-4
TS	186,000	5	-2	-2	+29	+26	+26
TP	177	9	-8	-9	+28	+18	+18
NH_3	2,140	1	-1	-1	+29	+29	+28
TKN	3,220	2	-1	-2	+29	+27	+27
Zn	20	40	-35	-35	+25	-20	-20
FC (counts/day	3.6×10^{13}	60	-50	-56	+22	-36	-42
BOD_5	3,700	11	-10	-10	+29	+19	+18
Pb	13	25	-23	-23	+31	-8	-15
Cu	9	10	-11	-11	+33	+11	+11
Average	-	18	-15	-16	+28	+7	+5

* Assumes Woodward Avenue WPCP is expanded to provide additional treatment capacity for population increases only.

SS,TP and TKN LOADINGS TO HAMILTON HARBOUR / COOTES PARADISE

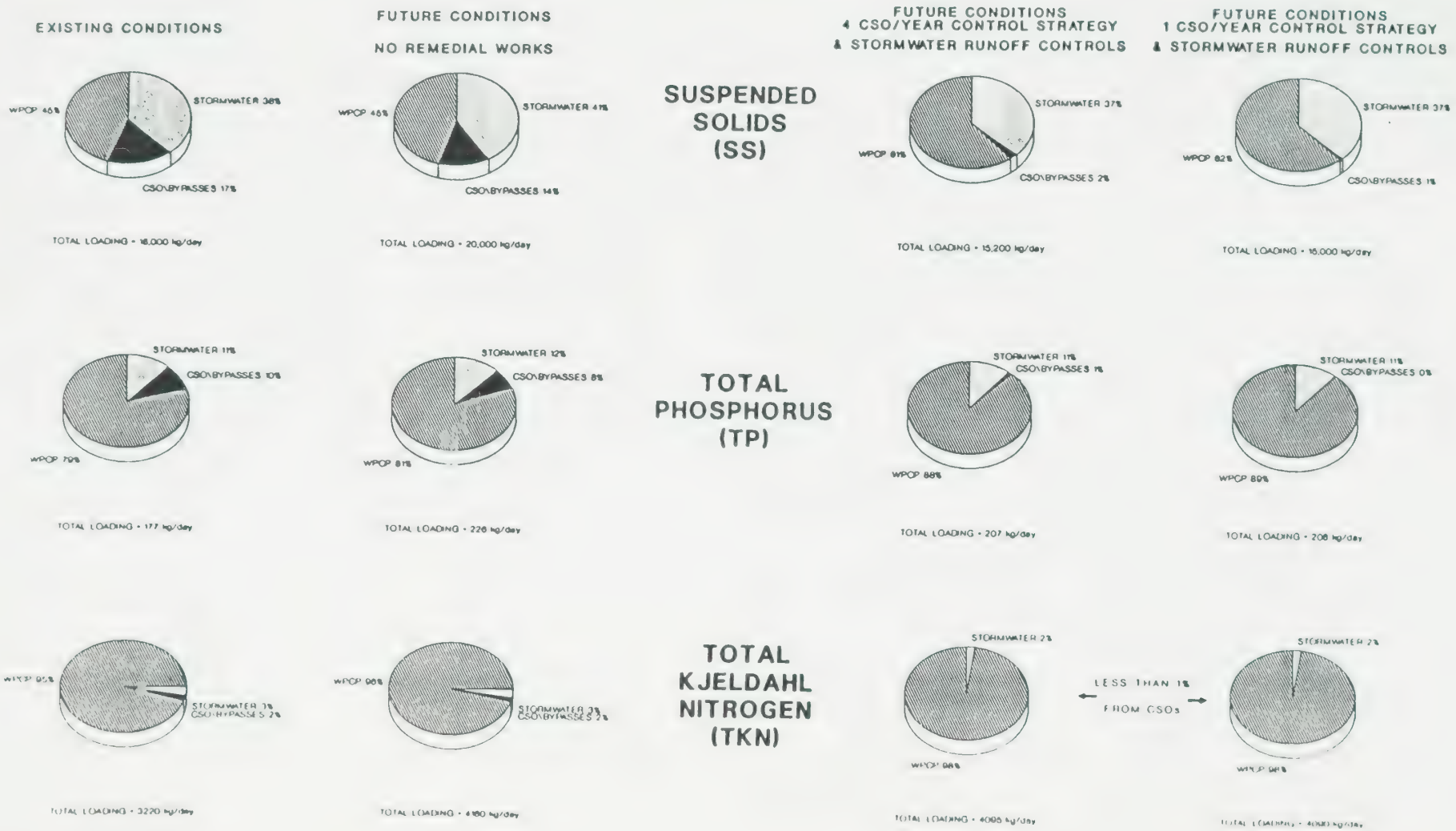


Figure 9-7

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4. Under existing development conditions total constituent loadings are expected to decrease approximately 15 percent under either CSO control strategy. Generally, the greater the contribution of CSO's to the existing loadings, the greater the reduction in the total loadings.

5. Under future conditions, with no CSO or stormwater remedial works, total constituent loadings are expected to increase approximately 28 percent. This increase in loadings is primarily due to increased inflows to the WPCP as a result of population increases.

6. Generally, if CSO's contribute more than 15 percent of the existing loadings, then loadings under future conditions are expected to be approximately 3 to 35 percent less with the recommended CSO, WPCP and stormwater remedial works. This trend is illustrated for SS, Zn, FC and Pb loadings in Table 9.9. Conversely, if CSO's currently contribute less than 15 percent of the existing loadings, then loadings under future conditions are expected to be approximately 10 to 30 percent greater. This trend is illustrated for TS, TP, NH_3, TKN, BOD_5 and Cu loadings (Table 9.9). The loadings in the latter case increase since the WPCP contributes the majority of the loadings and WPCP inflow volumes increase due to increased growth in the area.

7. Constituent loadings to Hamilton Harbour/Cootes Paradise under future conditions with the proposed remedial works are highly variable, ranging from a decrease in loadings of 42 percent, to an increase of approximately 29 percent. Overall, the general trend is for loadings to increase by approximately 6 percent, compared with a 28 percent increase if no remedial works are constructed. Further loading reductions could be attained by improving the quality of the Woodward Avenue WPCP effluent.

The preceding analysis, and the results in Section 9.5 clearly illustrate:

- the WPCP is generally the largest contributor of most constituents to Hamilton Harbour/Cootes Paradise;
- storm water and CSO loadings are often of the same order of magnitude; and
- total loadings are more dependent on the quality of effluent from the WPCP, than on whether a 1 or 4 CSO/year control strategy is adopted.

These observations tend to suggest CSO control may not be as important an issue as originally perceived. Before such judgements are made, however, it is useful to consider the points outlined below.

1. The public connects combined sewer overflows to the water quality degradation of Hamilton Harbour. For example, the general public connects CSO's to:
   - bacteria; and
   - floatables.

   Conversely, the general public does not normally perceive separated stormwater runoff to be the key source of pollutants.

2. Swimming in the west end of Hamilton Harbour is ultimately desired. Numerous CSO outfalls exist in the area and are assumed to be the largest contributor of fecal coliforms (FC) to the receiving waters. As FC levels are used to determine whether or not beaches remain open or are closed, CSO's will have to be reduced in the area in order to create a healthy recreational (i.e. body contact) environment.

3. The current study incorporated only common water quality parameters which are present in both separated stormwater runoff and combined sewage. Hamilton, however, has a diverse industrial environment and numerous hospital/health care facilities located throughout the combined sewer system. Consequently, other less common and potentially lethal compounds may be discharged to the combined sewer system on a regular basis, and to Hamilton Harbour via CSO's. Since these additional compounds may be removed from sanitary sewage by WPCP treatment processes, the capture and treatment of CSO's is important.

In summary, the reduction of CSO loadings to Hamilton Harbour/Cootes Paradise is an important issue which needs to be addressed if RAP objectives for the Harbour (i.e. swimming) are to be met. WPCP upgrades, to improve effluent quality, should also be considered in the future since the WPCP is the largest contributor of SS, TP, TKN, etc.

9.7 Total Cost of Proposed Works

Table 9.10 summarizes the total costs of the recommended remedial works to:

- reduce CSO's to 1 or 4 CSO events/year; and
- maintain current effluent quality at the Woodward Avenue WPCP during storage and treatment of CSO's.

The estimated costs range from approximately $190 to $250 million. These capital costs are equivalent to approximately $10 to $12.5 million dollars annually if a 20 year time frame is adopted for implementation.

TABLE 9.10

TOTAL COST OF RECOMMENDED CSO AND WPCP REMEDIAL WORKS

Level of Control	CSO Storage Option	Cost of CSO Storage (million)	WPCP Improvement Costs (million)	Estimated Total Cost (million)
4 CSO's/year	3. Offline storage facilities and inline storage from the Woodward Avenue WPCP to Gage Avenue (Figure 9.3).	95.8	90	186
4 CSO's/year	4. Offline storage facilities and inline storage from the Woodward Avenue WPCP to Wentworth Avenue (Figure 9.4).	106.1	90	196
1 CSO/year	3. Offline storage facilities and inline storage from the Woodward Avenue WPCP to Gage Avenue (Figure 9.3).	139.1	100	239
1 CSO/year	4. Offline storage facilities and inline storage from the Woodward Avenue WPCP to Wentworth Avenue (Figure 9.4).	147.6	100	248

10 - CONCLUSIONS AND RECOMMENDATIONS

10.1 Conclusions

Conclusions and recommendations which may be made as a result of the Region Municipality of Hamilton-Wentworth Pollution Control Plan study are outlined below.

1. A pollution control plan to reduce pollutant loadings to the local receiving water bodies has been developed for the Regional Municipality of Hamilton-Wentworth. The study area included the entire Region, with the City of Hamilton as the core study area. Approximately 50 percent of the area is serviced by combined sewers. Three WPCP's service the study area. The Woodward Avenue WPCP services Hamilton, Stoney Creek, Ancaster and Glanbrook, an area with a current population of approximately 380,000 and an ultimate population of 500,000 by 2025. Plants in Dundas and Waterdown service their respective communities.

2. Sources of contaminated discharges to the receiving water bodies include:
   - combined sewer overflows through 22 major CSO outfalls;
   - water pollution control plant bypasses and effluent; and
   - separated stormwater system discharges.

   Each of these sources were considered in the study, although CSO's and WPCP discharges were the main items of concern. The impact of treating CSO's at the WPCP was also assessed in detail.

3. The discharge of CSO's through the outfalls is largely controlled by flow regulators. Flow regulators are hydraulic structures in the combined sewer system which determine the quantity of combined sewage discharged to either the CSO outfalls or to the sanitary interceptor/WPCP. In excess of 100 flow regulators exist throughout the Region. Most regulators are located on local streets. This study identified approximately 35 CSO flow regulators which have a significant effect on the response of the sewer system during rainfall events.

4. WPCP analysis in this study centred on the Region's main plant located on Woodward Avenue. This conventional WPCP, which currently treats approximately 303 million litres per day (MLD), has design capacities of:

- 409 MLD for primary treatment; and
- secondary treatment capacities of 545 MLD (long term) to 615 MLD (short term).

5. The items below summarize the existing characteristics of CSO's within the Region:

i) Combined sewer overflows were observed, or are expected, at the CSO outfalls for small (i.e. 3 to 5 mm) events.

ii) Dry weather overflows of sanitary sewage were observed, or are expected, through the Wentworth and Birch CSO outfalls. These CSO's occur due to low flow regulator weir heights and/or insufficient capacity downstream of the flow regulators.

iii) On average, CSO's occur at the 22 major outfalls once every 8 days. Approximately 15, 38, 56 and 85 percent of the annual CSO volume is discharged from the top 1, 3, 5 and 10 outfalls, respectively. Therefore, approximately half of the outfalls contribute less than 15 percent of the annual CSO volume.

iv) CSO's are expected to discharge 4.38 million cubic metres to the receiving waters during a typical May 1 to October 31 season. Approximately one–third of this volume is discharged through outfalls located by human contact areas. The remaining CSO volume is generally discharged in the east end industrial areas.

v) CSO's, on the average, are composed of 7 percent sanitary sewage and 93 percent stormwater runoff.

vi) The Woodward Avenue WPCP is currently operated to maximize the volume of combined sewage which is treated during rainfall events. However, the primary treatment system is limited in that it can treat only 10 to 20 percent more sewage than dry weather flow conditions. This limitation requires bypassing excess flow around the primary treatment system. Primary bypassing is normally required for rainfall events greater than 5 mm in depth, or approximately 50 percent of all rainfall events.

The secondary treatment system has the capacity to treat approximately 50 (long–term) to 100 (short term) percent more sewage than dry weather flow conditions. Secondary bypassing is normally required for rainfall events greater than 10 mm or approximately 25 percent of all rainfall events.

vii) The existing combined sewer system and WPCP has sufficient capacity to convey and treat approximately 90 percent of all inflows to the system for a 5 mm event. Conversely, two-thirds of all inflows during a 2-hour 35 mm event (i.e. the 1 CSO/year design storm) will be routed to the receiving water bodies with little or no treatment. Substantial remedial works are, therefore, required to attain high levels of CSO control.

viii) The existing Greenhill Avenue CSO storage facility serves the majority of the "mountain's" combined sewer system. This facility is sufficient to store CSO's from rainfall events up to approximately 15 mm in depth, or approximately 85 percent of all rainfall events. CSO's to Redhill Creek from the Greenhill Avenue facility are expected to occur approximately 13 times from May 1 to October 31. CSO's exiting the facility have received primary treatment.

ix) Data gathered in this study and simulation modelling indicated several limitations in the Woodward Ave. WPCP which substantially decrease the ability of the present plant to handle stormflows. These limitations are:
   - the primary clarifiers are over capacity for normal dry weather flow in comparison to their design size, but still achieve substantial removal of suspended soils (ca 60%): even during wet weather surveys.
   - secondary clarifiers on the north side have substantial performance problems under storm surge conditions, prompting the operators to by pass the secondary units when this occurs. This phenomena was observed to be highly unstable and explained by inactive settling zones by the simulation model.
   - the ability of the operators to control the flow split between the north side and south side is limited; simulations indicated that control of the flow split could increase plant performance during storm flows.

6. Proposed development patterns in the Hamilton Harbour/Cootes Paradise drainage basin indicates the ratio of combined sewer services to separated sewer services will shift from 1:1 to 1:2.

7. On average, under existing conditions, CSO's, stormwater discharges and WPCP discharges account for 18, 24 and 58 percent of the total loadings to Hamilton Harbour/Cootes Paradise, respectively. With the projected population and development patterns in the area, these contributions are expected to change to 14, 27 and 59 percent, respectively. Overall,

the Woodward Avenue WPCP is, and is expected to remain, the largest contributor of most constituents to Hamilton Harbour/Cootes Paradise. Therefore, substantial reductions in constituent loadings will require the improvement of WPCP effluent quality. Under current conditions, CSO'S contribute the majority of Fecal Coliforms to the receiving waters.

8. With the projected population and development patterns, total constituent loadings to Hamilton Harbour/Cootes Paradise are expected to increase approximately 28 percent above existing conditions.

9. Previous RAP studies have identified the key water quality parameters affecting the receiving waters and the required loading reductions to achieve the desired water quality objectives. Three key parameters are suspended solids, total phosphorus and TKN. Loading reductions of approximately 90, 80 and 35 percent have been set for CSO's, WPCP effluent and stormwater discharges, respectively.

10. An assessment of CSO control strategies suggested appropriate levels of control, to meet RAP objectives, may be attained by adopting either a:
    - 4 CSO events/year control strategy; or a
    - 1 CSO event/year control strategy.

    These control strategies are frequency based and have been used in previous CSO studies. A 1 CSO event/year control strategy may be adopted in environmentally sensitive or human contact areas. Conversely, a 4 CSO events/year control strategy may be appropriate in industrial areas. An analysis of the 1 and 4 CSO events/year control strategies indicate they reduce CSO volumes and loadings by approximately 95 and 87 percent, respectively, for the typical year. These levels of control, therefore, bracket the 90 percent reduction proposed by RAP documents.

11. Numerous remedial options to reduce constituent loadings to the receiving waters using source controls, sewer system controls and WPCP modifications were evaluated. The evaluation was based on various factors including:

    - potential level of improvement;
    - technical feasibility;
    - cost–effectiveness;
    - implementation considerations;

- conflict potential;
- environmental assessment progress; and
- operation and maintenance considerations.

This process yielded a short list of recommended control alternatives which were subsequently classified as being either an:

- interim alternative; or a
- long term alternative.

12. Interim control alternatives include the following:

i) Within the sewer system, certain flow regulators may be adjusted to reduce CSO frequencies and volumes. Emphasis was placed on adjusting regulators upstream of human contact areas. Flow regulator adjustments include raising the height of the overflow weirs and/or resetting sluice gate positions. These adjustments are expected to reduce CSO frequencies by 0 to 40 percent and CSO volumes by 5 to 60 percent.

ii) At the WPCP, internal modification should allow the present plant to treat storm flows of the size of Hurricane Hugo without bypassing, for the present settings of the regulators.

Required modifications include:

- improved secondary clarifiers
- application of step feed
- operating the sludge residence time (SRT) at 7–8 days
- installing flow control devices and monitoring equipment to control the split in flow between the north side and south side

13. Recommended long–term control strategies include remedial works which result in either major or minor reductions in constituent loadings. Major reductions in CSO's are possible through the use of inline and offline storage of CSO's. Inline and offline facilities store CSO's, which would normally be discharged to the receiving water bodies, for treatment until the treatment capacity becomes available at a WPCP. Offline, or end of line storage facilities, generally involve the storage of overflows at or near the point of discharge in either above ground or below ground facilities. Inline storage involves the storage of CSO's within the sewer system, usually in large tunnel sewers. Five configurations of inline and offline storage facilities were reviewed in order to determine the most effective combination. Results are summarized in Table 10.1. Options 3 and 4 in Table 10.1 are favoured since:

TABLE 10.1

CSO STORAGE ALTERNATIVES AND ASSOCIATED COSTS

Option and Description	Number of Offline Facilities*	Length & Diameter of Inline Tunnel	4 CSO/Year Control Strategy				1 CSO/Year Control Strategy			
			Cons'n Costs		Land Costs	Total Costs **	Cons'n Costs		Land Costs	Total Costs **
			Tanks	Tunnel			Tanks	Tunnel		
1. Separate Offline Storage Facilities at each outfall	22	-	72.1	0	4.0	76.1	115.3	0	5.8	121.1
2. Offline Storage Facilities with some Lumped Facilities	18	-	70.5	0	3.7	74.2	113.8	0	5.2	119.0
3. Offline Storage Facilities and Inline Storage from the Woodward Avenue WPCP to Gage Avenue	12	Length = 4.5 km D=7.0m for 4 CSO's D=9.0m for 1 CSO	42.5	52.0	1.3	95.8	72.3	65.0	1.8	139.1
4. Offline Storage Facilities and Inline Storage from the Woodward Avenue WPCP to Wentworth Avenue	10	Length = 6.2 km D=7.0m for 4 CSO's D=8.5m for 1 CSO	34.2	71.0	0.9	106.1	60.0	86.8	0.8	147.6
5. Offline Storage Facilities and Inline Stroage from the Woodward Avenue WPCP to Highway 403	9	Length = 11.2 km D=6.0m for 4 CSO's D=7.0m for 1 CSO	24.0	112.0	0.3	136.3	44.7	129.0	0.9	174.6

* Excluding the existing Greenhill CSO Storage Facility.
All costs in Million Dollars. Cost estimate data is included in Appendix E.

** Excludes WPCP expansion costs of $90 and $100 million for the 4 and 1 CSO/year control strategies, respectively.

- land is not required in Hamilton's highly developed east end (i.e. east of Gage Avenue) for offline storage facilities;
- the use of an inline tunnel west of the Woodward Avenue WPCP provides an alternate route for sewage flows to be conveyed to the WPCP, should the existing western interceptor be in need of repair; and
- capital costs are not as excessive as Option 5.

14. The proposed long-term modification to the Woodward Avenue WPCP is basically one of plant expansion:

The recommended expansions include those items listed below for the 1 CSO/year strategy:

a) Add two pumps to the pumphouse to bring it its full complement.

b) Expand the primary clarifiers by 2.4 times (i.e. add primary clarifiers equivalent to 1.4 times the existing primary clarifiers).

c) Expand the second clarifiers by 1.6 times (i.e. add new secondary clarifiers equivalent to 0.6 times the existing secondary clarifiers).

d) Expand the secondary aeration tanks by doubling the south side (i.e. provide additional aeration tanks equivalent in size to the present south size aeration tanks).

e) Add a new detritor unit in parallel to the existing detritor unit; connect this unit to the existing pumphouse in parallel with the existing detritor unit and then connect it to the new primary tanks.

f) Provide an equalization tank at the WPCP, in conjunction with Storage Options 3 or 4, equivalent to the volume of storm water directed to the plant and not stored in the CSO facilities (160 mL for a one overflow event). The equalization tank is required to control the storm surge as it arrives at the plant.

This option should meet existing C. of A. requirements for the plant, allow operating flexibility, improve nitrification in the effluent, handle anticipated dry weather flows from a population of 500,000 people, and treat normal dry weather flows plus stored CSO volumes from a one overflow event/season rainfall. It is estimated to cost approximately $100 million.

For the 4 CSO/year strategy, the estimated cost is 90 million for WPCP expansion. Expansion requirements are similar to those for the 1 CSO/year strategy, with smaller sizes required for the primary and secondary clarifiers and the equalizers tank.

Before implementing these works at the WPCP, some interim improvements in plant performance can be obtained by using step–feed and improving the secondary clarifiers on the north side. However, new construction of primary clarifiers can be anticipated at an early date, as a minimum requirement. Overall substantial retrofitting will be required at the WPCP to implement these works.

There is sufficient land available at the WPCP for this degree of expansion.

15. Long–term control strategies for reducing constituent loadings from separated stormwater systems centre on the use of stormwater management (wet) ponds to treat runoff from new developments. Wet ponds are passive devices which remove contaminants from stormwater runoff by settling solids and chemicals (e.g. heavy metals) attached to the solids. Wet ponds are expected to remove 80 percent of SS, 50 percent of TP, and substantial quantities of most other constituents as well.

16. Total costs of approximately $190 to $250 million have been estimated for the 4 and 1 CS0 events/year control strategies, respectively. The proposed works will;

    - store CSO's in either inline or offline facilities (Options 3 or 4, Table 10.1); and
    - treat the stored CSO's at the Woodward Avenue WPCP while maintaining the current level of effluent quality.

17. Minor reduction in CSO loadings may also be attained through:

    - roof downspout disconnection;
    - the use of swirl concentrators in conjunction with storage facilities;
    - Real Time Control of the combined sewer system; and
    - public education programs.

Roof downspout disconnection reduces CSO volumes by approximately 5 percent since runoff volumes entering the combined sewer system are reduced. Since roof downspout disconnection does not provide significant CSO reduction, and since it requires the

participation of the public, it is best suited for application where existing sewer system problems exist (i.e. areas with basement flooding).

Real Time Control of the Region's combined sewer system, and of any of the proposed storage facilities, may be an effective means of reducing CSO frequencies and volumes. The use of Real Time Control was assessed qualitatively here as the Region is investigating this technology elsewhere. Other studies have suggested Real Time Control may reduce the capital costs associated with CSO control by 15 to 30 percent.

Swirl concentrators can be used to reduce CSO's, however, they may have to be used with storage facilities since:

- they tend to remove only 20 to 50 percent of incoming suspended solids; while
- RAP documents suggest a 90 percent reduction in CSO constituents are required to meet water quality objectives.

Additional research is, therefore, required before stand alone swirl concentrators can be recommended as an effective means to reduce CSO's.

Public education programs may also provide small reduction in loadings to the receiving waters and are usually cost effective.

18. Under existing and future conditions the recommended CSO remedial works are expected to decrease CSO loadings by 87 percent for a 4 CSO/year control strategy, and 95 percent for a 1 CSO/year control strategy.

19. Under existing conditions with the proposed CSO and WPCP remedial works, loadings from the WPCP are expected to increase approximately 3 percent. For future conditions, loadings from the WPCP are expected to increase approximately 33 percent. These increases in loadings result from the increase in WPCP inflow due to CSO diversions and future development.

20. Without stormwater management (wet) ponds in future developments, loadings from separated stormwater systems are expected in increase 40 percent. With wet ponds, the 1``loadings are expected to increase only 15 percent.

21. Under existing conditions, total loadings are expected to decrease approximately 15 percent with the recommended remedial works. For future conditions, with the recommended works, total loadings are expected to increase approximately 6 percent, compared to a 28 percent increase with no remedial works. The increase for future conditions is primarily the result of increased WPCP flows due to population growth in the area. Improving the quality of the Woodward Avenue WPCP effluent would reduce loadings further.

22. Loadings of the three key water quality parameters as defined by RAP (suspended solids, total phosphorus and TKN) under existing and future conditions are illustrated in Figure 10.1.

10.2 Recommendations

1. Dry weather overflows of sanitary sewage from the combined sewer system should be eliminated.

2. In the interim, consideration should be given to adjusting the overflow regulators reviewed in Section 9 and Appendix B to reduce the frequency and volumes of CSO's at six outfalls.

3. The recommended long–term control strategy is based on:

   - storing CSO's in either inline and/or offline facilities;
   - treating the stored combined sewage at the Woodward Avenue WPCP; and
   - treating stormwater runoff from future development in stormwater management (wet) ponds.

   This control strategy has been recommended because:

   - inline and offline storage of CSO's have proven to be cost effective in reducing CSO's;
   - treating the stored CSO's at an expanded Woodward Avenue WPCP allows CSO treatment facilities to be centralized, thereby reducing capital, operation and maintenance costs; and
   - stormwater management (wet) ponds are an effective means of treating runoff and they may be easily incorporated in future developments.

4. Consideration should be given to using public education programs, swirl concentrators and Real Time Control techniques to reduce CSO's. Public education programs, although not

SS,TP and TKN LOADINGS TO HAMILTON HARBOUR / COOTES PARADISE

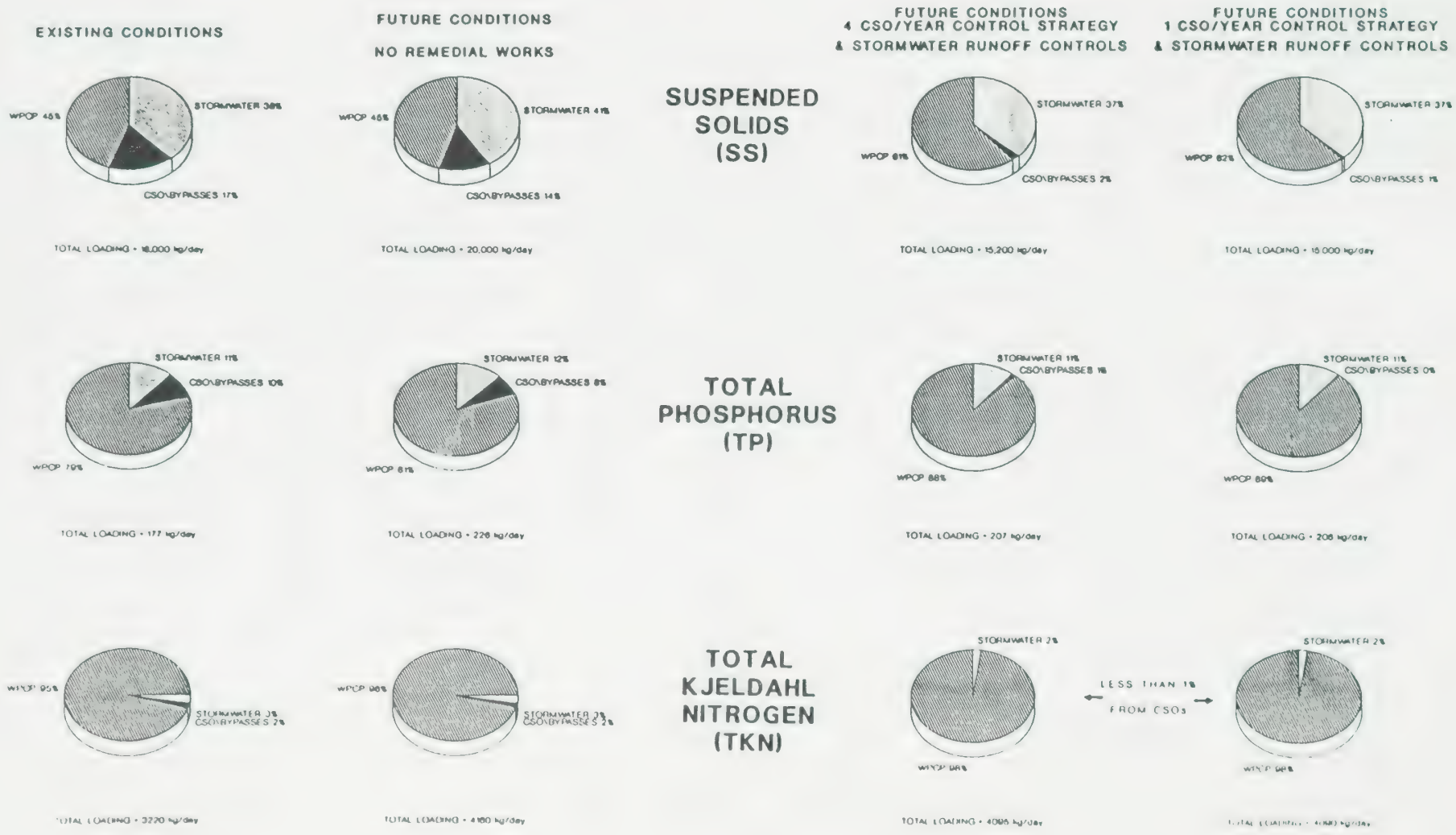

Figure 10-1

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providing significant reduction in CSO's, are usually cost effective. Swirl concentrators are recommended as they may be used in conjunction with storage facilities to decrease the size, and hence cost, of the facilities. Current and future studies of swirl concentrators should be referred to further define the feasibility of this option.

Real Time Control techniques are also recommended since they may be used to optimize system performance. Other studies have suggested Real Time Control may reduce the capital costs associated with CSO control by approximately 15 to 30 percent.

5. The development of a Remedial Action Plan to reduce constituent loadings to the receiving waters should include a means for prioritizing the outfalls where CSO controls are to be established. Prioritizing the outfalls should help to determine where remedial works are required most. Criteria which may be used to prioritize an outfall include:

   - the volume of CSO discharged;
   - the frequency of CSO's;
   - the proximity of the outfall to human contact areas;
   - the expected constituents in the sanitary sewage (i.e. what industries are upstream of the outfall?)
   - the cost of controlling CSO's at the outfall; and
   - the ease with which the CSO controls may be installed.

6. A detailed investigation comparing the area covered by storm cells and the area draining to common/lumped CSO storage facilities should be conducted in order to optimize the size of the facilities. It is expected that such an investigation may allow the size of the facilities to be decreased, resulting in substantial cost savings.

7. Drainage patterns, and hence design storm CSO storage volumes, for each CSO outfall should be verified prior to detailed design as numerous simplifications were made in this study.

8. It is recommended that the following steps be carried out over the next 5 year time frame to develop an improved WPCP expansion plan to that developed in this study.

   1. Evaluation plant improvements possible from the ongoing studies (1991).

2. Based upon additional calibration data, and refined objectives for plant performances examine the simulation model calibration and adjust plant expansion requirements into several time frames, for example:

   0 – 5 years

   5 – 10 years

   10 – 15 years

   15 – 20 years

   20 – 25 years

   Expansion of the plant should consider the peaking of the influent hydrograph until system storage units or equalization tanks are built (this could be done in 1991 to 1992).

3. Initiate detailed studies of other treatment options (1991).

4. Develop a plan for plant improvements to the present plant.

5. Develop a detailed plan for plant expansion. We see this to involve the following elements:
   - additional primary clarifiers fairly immediately, since the present clarifiers are already overloaded
   - evaluate the performance of additional clarifiers and determine whether a biological treatment strategy for storm flows is the preferred option, or whether physical-chemical options should be used
   - determine whether additional aeration tanks and final clarifiers should be built, and if so, their size.

9. Other technologies for treatment of CSO volumes instead of biological treatment options include coarse roughing filters or small phys–chem plants (coagulation – flocculation – settling). These are promising alternatives but pilot scale and full scale demonstration are required before they can be considered in a pollution control plant.

10. Decisions upon the usefulness of the Dundas WPCP for stormwater control should be made in the near future after simulation studies and full scale step feed work have been completed and evaluated.

11. A detailed plan for retrofitting water quality control devices into the remaining urban areas should be developed in the future after a significant portion of this pollution control plan has

been completed and its contribution to improving Hamilton Harbour water quality has been measured. As opportunities for retrofitting occur due to redevelopment, installation of passive technologies (wet ponds) or provision for active technologies for stormwater treatment should be considered.

12. This report contains planning level estimates of constituent loadings to Hamilton Harbour. If it is required to increase the accuracy of the constituent loadings to Hamilton Harbour, then it is recommended that a long–term CSO quantity/quality monitoring program be adopted. The program should span at least one calender year so that CSO quantity/quality characteristics for various types of runoff events (e.g. melt events and thunderstorms, etc.) could be determined. Large runoff events (e.g. greater than 12 mm) are the key events that would have to be monitored during the program since such events are estimated to contribute at least 60 percent of the CSO volume/loading to Hamilton Harbour. It is also recommended that CSO quantity/quality monitoring be conducted at a minimum of 4 outfalls to determine representative CSO characteristics throughout the Region.

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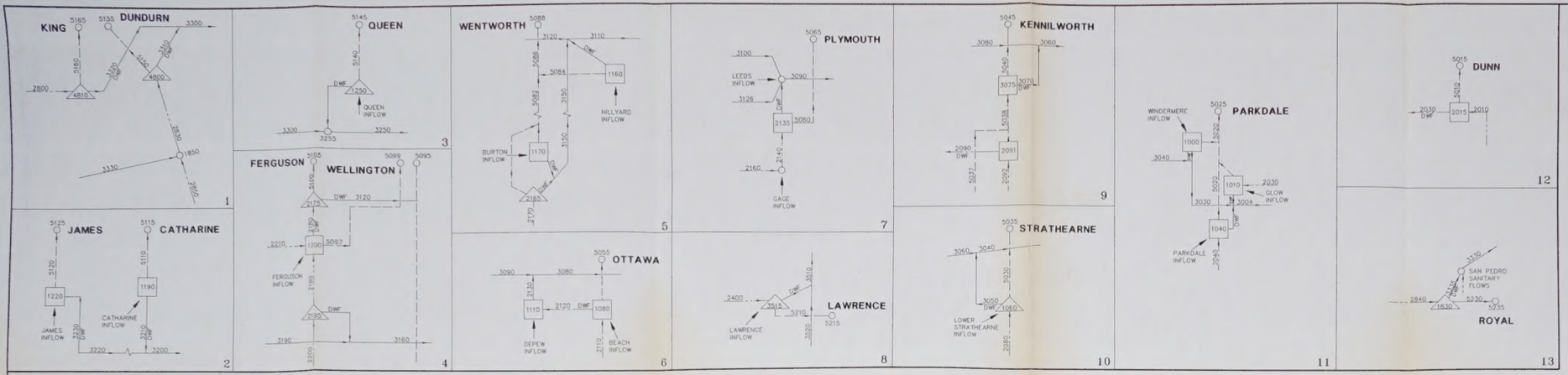

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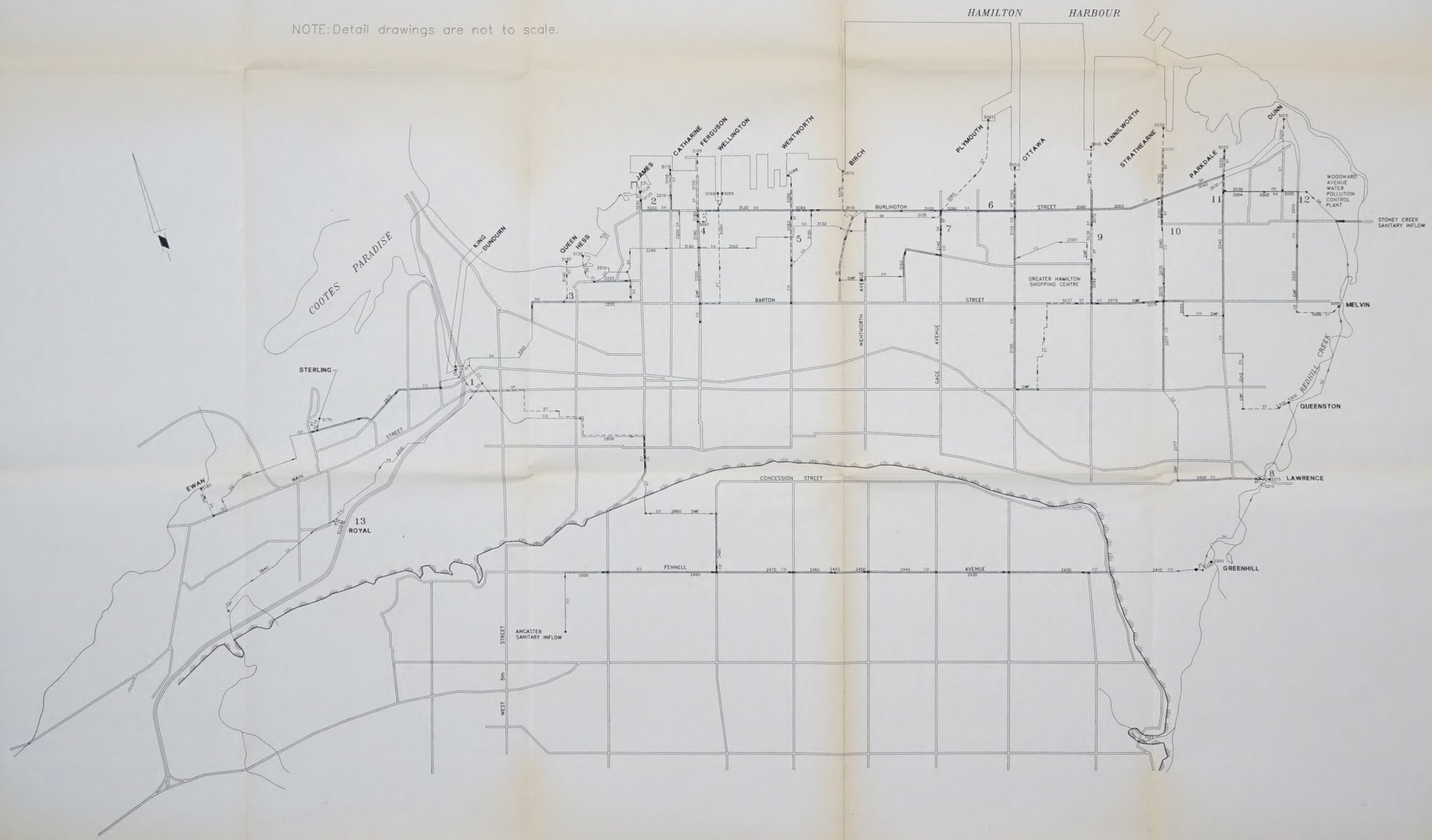

LEGEND

- Sanitary Sewer
- Storm Sewer
- Combined Sewer
- Manhole or Subcatchment Inflow Point
- Automatic Flow Regulator
- WPCP Controlled Flow Regulator
- Direction of Flow
- Direction of Dry Weather Flow
- MELVIN — CSO Outfall

Scale: 1:18000

REGIONAL MUNICIPALITY OF HAMILTON - WENTWORTH

POLLUTION CONTROL PLAN

COMBINED SEWER DRAINAGE BASIN MODELLED SEWER ELEMENTS

DRAWING NO. 88143-B-2

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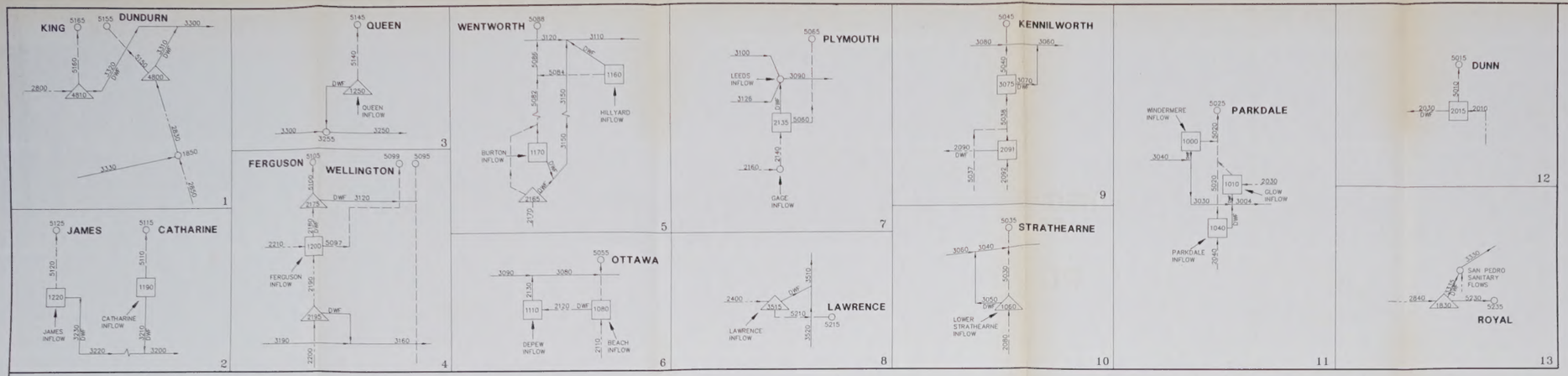
REGIONAL MUNICIPALITY OF HAMILTON - WENTWORTH

POLLUTION CONTROL PLAN



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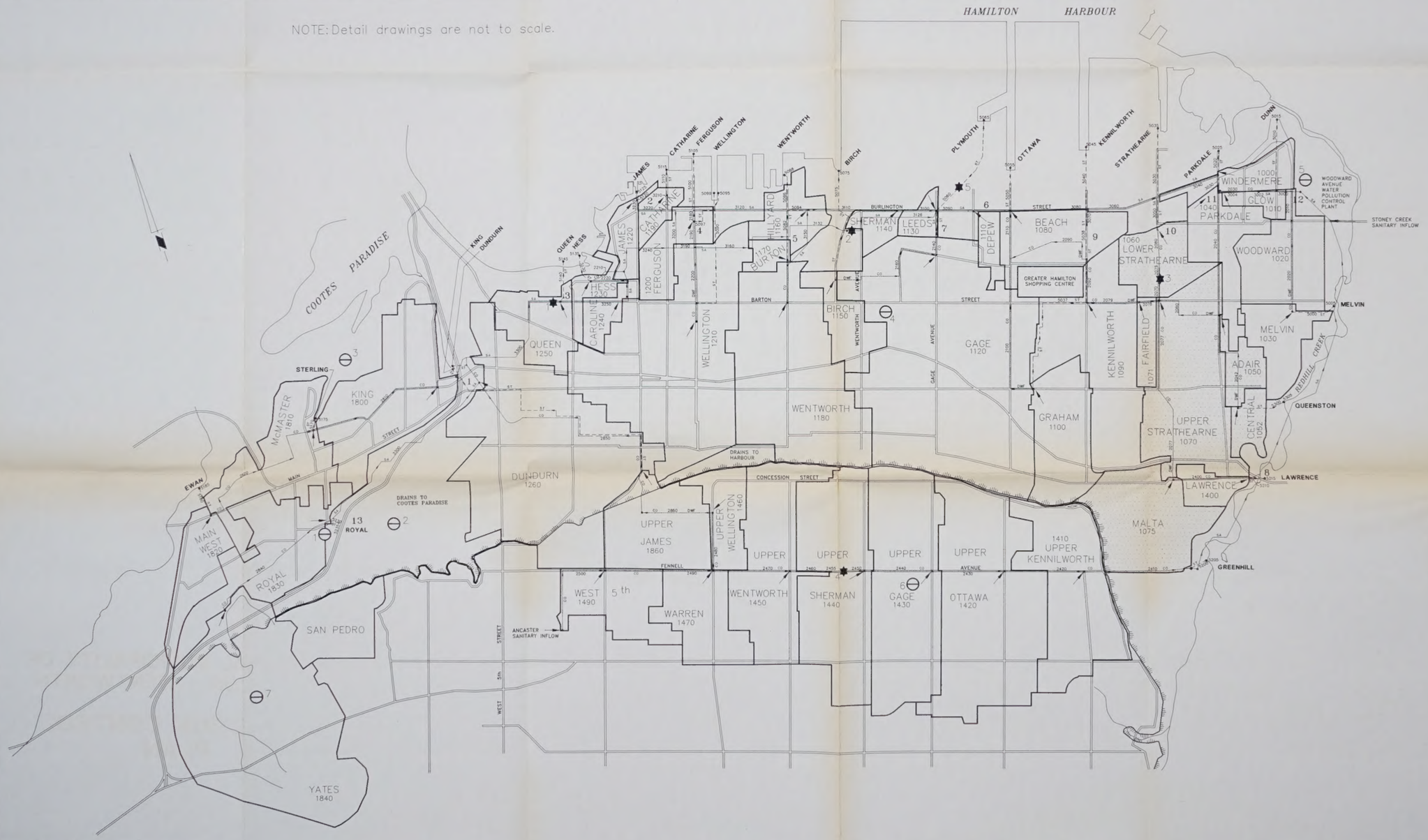

LEGEND

- Subcatchment Limits
- QUEEN — Subcatchment Name
- 1250 — Subcatchment Number and Inlet Number
- Subcatchment Inflow Point
- Sanitary Sewer
- Storm Sewer
- Combined Sewer
- Manhole or Subcatchment Inflow Point
- Automatic Flow Regulator
- WPCP Controlled Flow Regulator
- Direction of Flow
- Direction of Dry Weather Flow
- Rainfall Gauging Station
- Flow Monitor
- MELVIN — CSO Outfall
- Representative Combined Sewershed

Scale: 1:18000

COMBINED SEWER DRAINAGE BASIN SUBCATCHMENTS AND SEWER SYSTEMS

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